Comparative Construction Costs of Typical Low-rise Office Buildings in South Africa

by

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Declaration

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Summary

Low-rise multi-storey office buildings are required globally on a regular basis, and have thus become an increasingly important field within the engineering industry. A critical decision that has to be made during the initial stages of the planning and design of such a building is the selection of the structural framing material. This decision typically involves choosing between steel and reinforced concrete and is important as it will influence both the construction time and cost.

In South Africa, concrete is currently the default option for the framing material of multi-storey office buildings. This is in contrast to some other parts of the world where steel frames hold a dominant market share. Very little information on the costs and construction time could be traced which provided a comparison of steel and concrete framed office structures in South Africa. This study aims to fill this knowledge gap by providing a cost comparison between steel and concrete structural alternatives for the structure of a typical low-rise office building.

A building configuration was developed that can be seen as being representative of a typical low-rise office building in South Africa. Its structure was then designed employing various steel and concrete structural alternatives. Several meetings were held with industry professionals during the study to obtain insight into current South African construction practices, including steel fabricator's preferences, construction times and costs, and designer's preferences to name a few.

The construction programmes for each of the structural alternatives were developed in detail and compared. It was shown that the steel structures were able to offer a shorter construction time of approximately one month when compared to the concrete structures. This equates to a reduction of 12.5 % in the total construction time. A detailed cost comparison was developed using 2016 South African construction costs in the Western Cape. In addition, the influence of time-related costs, such as lower preliminary and general (P&G) costs, reduced interest charges and the ability to earn income at an earlier stage, were all incorporated into the cost comparison. Thereafter, a sensitivity analysis was performed to better understand how changing various parameters would influence the cost model.

The study revealed that for a typical low-rise office building constructed in South Africa, a steel framed structure supporting hollowcore units acting compositely with the steel beams, provided the most cost-effective solution. The difference between the cheapest steel and concrete structural alternatives was found to be 0.6 % of the total construction cost, which equated to a cost difference of approximately R 150 000.00. Furthermore, the study presented a methodology

for developing cost comparisons between different structural alternatives, and highlighted the importance of considering time-related costs when doing a cost comparison.

Opsomming

Lae multi-verdieping kantoorgeboue word wêreldwyd op 'n gereelde basis vereis, die gevolge hiervan lei tot 'n verhoging in belang vir die veld in die ingenieursbedryf. Vir die toepaslike gebou, is die keuse van die strukturele raammateriaal 'n noodsaaklike besluit in die eerste fase van die beplanning en ontwerp proses. Hierdie besluit behels tipies die keuse tussen staal en gewapende beton, belangrik aangesien dit beide die konstruksietyd en koste sal beïnvloed.

Tans in Suid-Afrika se ingenieursindustrie word beton bo staal vir die raammateriaal van 'n multi-verdieping kantoorgebou verkies. Dit is in teenstelling met sommige ander dele van die wêreld waar staalrame 'n dominante markaandeel hou. Daar is bevind dat daar 'n tekort aan inligting is, wat die vergelyking tussen die koste en konstruksie tyd vir staal en beton geraamde strukture in 'n Suid-Afrikaanse konteks bemoeilik. Dié studie dien as 'n poging om die leemte in kennis te vul, en 'n koste vergelyking tussen staal en beton strukturele alternatiewe vir 'n tipiese lae-styging kantoorgebou.

'n Gebou uitleg is ontwikkel wat as verteenwoordigend van 'n tipiese lae multi-verdieping gebou in Suid-Afrika beskou kan word. Die verteenwoordigende struktuur was verder ontwikkel deur van verskeie staal en beton strukturele alternatiewe gebruik te maak. 'n Reeks vergaderings is met industriële deskundiges onderneem, om insig in die huidige Suid-Afrikaanse konstruksiebedryf te verkry insluitend, die voorkeur van staal vervaardigers en ontwerpers, konstruksie tye en pryse, om 'n paar te noem.

Vir elk van die alternatiewe strukture, word die konstruksie programme in detail ontwikkel en met mekaar vergelyk. Daar is gevind dat die staalstrukture in staat was om 'n korter konstruksie tyd, van ongeveer 'n maand, in vergelyking met die betonstrukture te bied. Dit is gelykstaande aan 'n verskil van 12.5 % in totale konstruksie tyd. 'n Gedetailleerde koste vergelyking is, met behulp van 2016 Suid-Afrikaanse boukostes in die Wes-Kaap, ontwikkel. Daarbenewens, was die invloed van tyd-verwante kostes, soos laer voorlopige en algemene (V&A) koste, verminderde rente en die vermoë om inkomste op 'n vroeër stadium te verdien ingesluit in die koste vergelyking. Daarna is 'n sensitiwiteitsanalise uitgevoer, om 'n beter begrip oor hoe die verandering van verskeie parameters die kostemodel sal beïnvloed te kry.

Die studie toon dat 'n tipiese lae multi-verdieping kantoorgebou, gebou in Suid-Afrika, 'n staal raam struktuur, wat hol-kern blaaie saamgestel ondersteun deur die staal balke die mees koste-effektiewe opsie is. Die verskil tussen die goedkoopste staal en beton strukturele alternatiewe is gevind as 0.6 % van die totale boukostes, wat gelykstaande is aan 'n koste verskil van ongeveer R 150 000.00. Dié studie lig verder 'n metode vir die ontwikkeling van koste vergelykings tussen

verskillende strukturele alternatiewe, en beklemtoon die belangrikheid deur tyd-verwante kostes te oorweeg wanneer 'n koste vergelyking ondersoek word.

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Contents

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Nomenclature

Symbols

Symbol	Units	Description
L	m	Length
-	kPa	Pressure
-	MPa	Pressure
T	°C	Temperature
Δ	mm	Deflection
f	Hz	Frequency
H_p	m	Heated perimeter
A	m^2	Cross sectional area

Acronyms

Acronym	Description
AISC	American Institute of Steel Construction
ASI	Australian Institute of Steel Construction
BCSA	British Constructional Steelwork Association
CISC	Canadian Institute of Steel Construction
FLS	Fire Limit State
P&G	Preliminary and general
PT	Post-tensioned
RC	Reinforced concrete
SANS	South African Nation Standard

SCI	Steel Construction Institute
SLS	Serviceability Limit State
SPM	Slab Panel Method
ULS	Ultimate Limit State
USA	United States of America

Chapter 1

Introduction

1.1 Subject

The subject of this thesis is a cost comparison between various steel and concrete structural alternatives for the structure of a typical low-rise office building in South Africa.

1.2 Background

The past century has seen a great increase in the need to provide floor space in the form of multi-storey buildings (Mathys, 2003). The design of multi-storey structures has thus become an increasingly important field within the global engineering industry. To illustrate this point, it can be pointed out that commercial buildings, which include, amongst other things, offices, shopping outlets and mixed residential-commercial buildings, currently account for 20 % of the construction output in the European Union (Davison, 2012). This equates to over 20 million square metres of floor space per year. A variety of structural systems are currently employed in the design and construction of these structures, and it is quite striking to note the diverse preferences that exist in different parts of the world with regards to which structural system to implement on a project.

A primary decision that has to be made during the initial stages of a project is the selection of the structural framing material. For the overwhelming majority of multi-storey office buildings this decision involves a selection between steel and reinforced concrete. It is important that the best material is chosen at an early stage of the project because it is unlikely that this decision will be changed. Such a change can have significant implications with regards to construction cost and programme and can influence the design of other major structural elements, such as the foundations, cladding and finishes (Barrett Byrd Associates, 2016). Steel and concrete present different advantages and disadvantages when used to construct the frame of a multi-storey building, and it is important that these are understood so that the most suitable framing material can be selected as early in the project as possible.

In South Africa, reinforced concrete is currently, and has been for some time, the default option when selecting a framing material for multi-storey office buildings. As a result, there are currently very few office buildings constructed with a steel frame in this country. This decision is primarily driven by the current perception that a concrete structure is cheaper than the equivalent steel structure. This perception is in contrast to some other parts of the world, where steel frames currently hold a dominant share of the multi-storey construction market. In the United Kingdom, for example, steel currently dominates the market, and has done so for the past 30 years. In 2015, steel frames captured a share of 68 % of the multi-storey office building market, compared to a share of just 23 % for in-situ concrete buildings (Tata Steel, SCI and BCSA, 2015a).

Similarly, in the United States of America (USA) steel holds a dominant market share. A study undertaken by the American Institute of Steel Construction (AISC), revealed steel to be the leading framing material for non-residential multi-storey construction in the USA, with a market share of 55 % compared to only 21 % for reinforced concrete (American Institute of Steel Construction, 2012).

The perception that concrete structures are cheaper than steel structures leads to the self-perpetuating cycle that is shown in Figure 1.1 below:

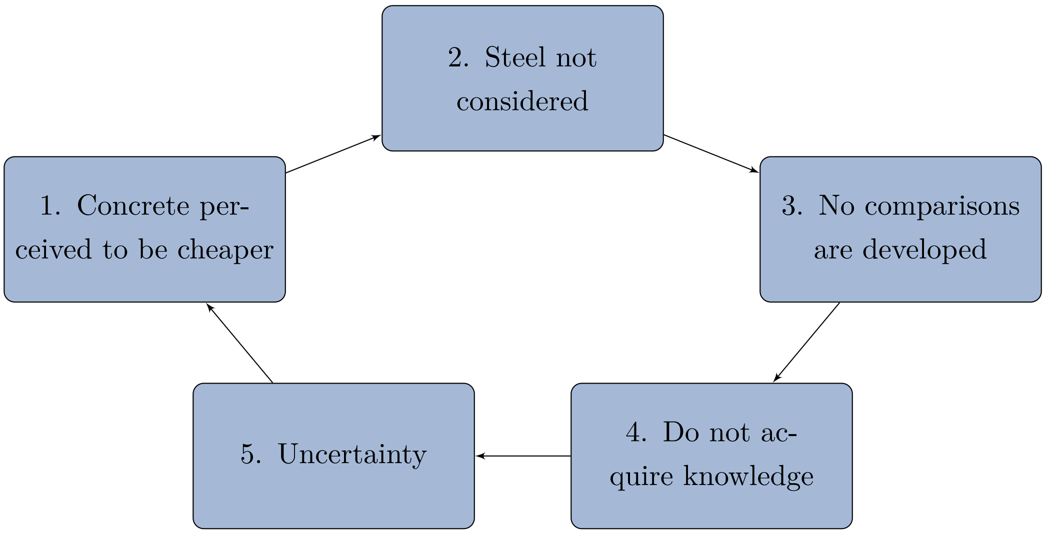


Figure 1.1: Self perpetuating cycle in South Africa

The aforesaid perception results in steel not always being given any consideration. Therefore, office buildings in South Africa are very rarely constructed using steel, and few comparisons are developed between steel and concrete framed structures. Additionally, the infrequency of steel building design and construction leads to a dearth of steel knowledge and experience in South Africa. This, in turn, results in uncertainty regarding the actual cost-effectiveness of steel and concrete buildings and how they compare with one another.

One of the possible reasons why steel framing systems are not given sufficient consideration when selecting a framing material can be attributed to the lack of readily available comparative information regarding different framing systems. At present, no comparison between steel and concrete structural alternatives for multi-storey office buildings in South Africa could be traced from literature or any other source. This means that professional teams will be uncertain as to how various steel and concrete structural alternatives compare with one another, and what may be the best solution to select for a specific project. Furthermore, the decision of selecting a building's framing material is often made in terms of limited criteria. Cost benefits that can be associated with a shorter construction programme, which steel projects would potentially offer, are rarely considered when comparing the cost of structural alternatives.

The absence of any meaningful comparative information means that even if steel were more cost-effective than concrete, this would not be known to professionals in the industry. Due to the lack of information, it is conceivable that the South African building industry could be missing out on a potential cost saving opportunity.

1.3 Purpose of the study

The purpose of this study is to compare the cost of a typical low-rise multi-storey office building when constructed using a variety of steel and concrete structural alternatives, and to establish whether steel structures may be the more economical option.

The study aims to compare the cost-effectiveness of steel and concrete structures, and reveal under what conditions, if any, steel should be considered. Furthermore, the study is intended to provide information to property owners, quantity surveyors, engineers and architects on the cost of different structural alternatives, and to demonstrate a methodology for developing comparisons that take into account key cost factors. This methodology should be able to assist project teams when selecting a structural solution to implement on a project.

1.4 Objectives

With the ultimate purpose of this study being to provide a comparison between steel and concrete structural alternatives for a typical low-rise multi-storey office building, the following objectives had to be achieved:

- Define a typical South African low-rise multi-storey office building and identify the key requirements dictating the form and configuration of such a structure.
- Develop a floor layout and building configuration that can be seen as being representative of a typical low-rise multi-storey office building in South Africa. This includes specifying aspects such as column positions, slab edges and floor-to-floor heights to name a few.
- Identify steel and concrete structural alternatives that are suitable for the building configuration that has been developed, and are representative of steel and concrete structural alternatives that are generally available to project teams in South Africa.
- Develop a detailed design for each of the structural alternatives, and identify key aspects requiring consideration during the design of these structures.

- Develop a construction programme for each of the structural alternatives.
- Do a cost comparison using current South African construction costs, and compare the cost-effectiveness of each of the different steel and concrete structural alternatives.
- Perform a sensitivity analysis to investigate the influence that changing various parameters would have on the cost comparisons that have been developed.
- Reveal under what conditions, if any, steel structural alternatives should be considered when selecting a structural solution to implement in a project.

1.5 Scope and limitations

The following boundaries were placed on the research conducted in this study:

- **Building layout** - The building layout employed in this study is based on what can be regarded as being representative of a "typical" (i.e. one that is frequently encountered) layout of a low-rise office building in South Africa. The same floor layout and column positions were used for each of the structural systems. For the steel structural alternatives, a long span layout is also considered by removing the internal columns, while all other column positions remain unchanged.
- **Low-rise versus tall structures** - This study did not investigate the influence that a change in the number of floors would have on the results of the cost comparison. The layout that has been used consists of a ground level, with three levels of suspended floors and a roof above ground. Tall multi-storey structures are typically considered to possess more than 20 stories and the design principles employed in such structures are quite different to those governing low to medium rise structures (Tata Steel, SCI and BCSA, 2015a). Tall buildings are thus excluded from this study.
- **Structural alternatives** - Only a limited number of structural alternatives could be investigated in this study. These alternatives included both a steel and concrete framed structure, with two different floor systems being considered within each framing system. The steel structural alternatives consisted of a steel frame supporting a composite Bond-Dek floor system, and a steel frame supporting precast hollowcore units. The concrete framed building options consisted of a reinforced concrete frame with a reinforced concrete flat slab floor, and a reinforced concrete frame with a flat slab post-tensioned concrete floor. The floor systems were selected as being representative of typical floor systems that are available to project teams in South Africa when choosing a structural system to implement in a project. There are additional floor systems that could have been considered for the comparison, however time did not allow for additional alternatives to to be considered in this study.
- **Non-structural frame components remain constant** - The study was limited to those components of the building that vary between each of the structural alternatives, namely

the building foundations and structural frame. Components which do not form part of the structural frame, such as the cladding, finishes, services and the roof, were assumed to be the same for each of the structural alternatives.

- **Sound design** - The focus of the research was not on developing an optimal solution for a typical low-rise office building, but rather on developing a "good" or sound design for each of the structural options that were considered.
- **Life cycle costs not considered** - The evaluation of the life cycle costs for the various steel and concrete structural alternatives fell outside the scope of this thesis. The life cycle costs that can be associated with the various building types were therefore not included in the cost comparison developed in this study.

1.6 Methodology of study

Figure 1.2 provides a graphical representation of the methodology that was followed during this study.

Initially, a literature study was performed in order to identify any previous comparative studies that have been developed, both in South Africa and internationally (Chapter 2). In addition to this, numerous meetings were held with professionals in the South African building industry in order to investigate existing preferences relating to the design and construction of multi-storey office buildings.

Following the completion of the literature study, and using knowledge gained from meetings, a layout was developed that could be seen as being representative of a typical low-rise office building in South Africa. The building was then designed employing a variety of steel and concrete structural alternatives, that are currently available in South Africa (Chapter 3).

The comparison of the structural alternatives was carried out in three main areas. The first area involved developing a construction programme for each of the steel and concrete structural solutions (Chapter 4). Next, the construction costs for each of the structural alternatives were calculated using current South African construction rates in the Western Cape. The cost implications associated with the varying durations of the construction programmes were incorporated into the cost comparison (Chapter 5). Finally, a sensitivity analysis was carried out in order to gain a better understanding of how varying certain parameters of the cost model would influence the cost comparison developed in this study (Chapter 6).

Considering the results of all three areas of comparison between the structural alternatives, conclusions could be made regarding the cost-effectiveness of various steel and concrete structural alternatives when used for the structure of a typical low-rise office building in South Africa (Chapter 7).

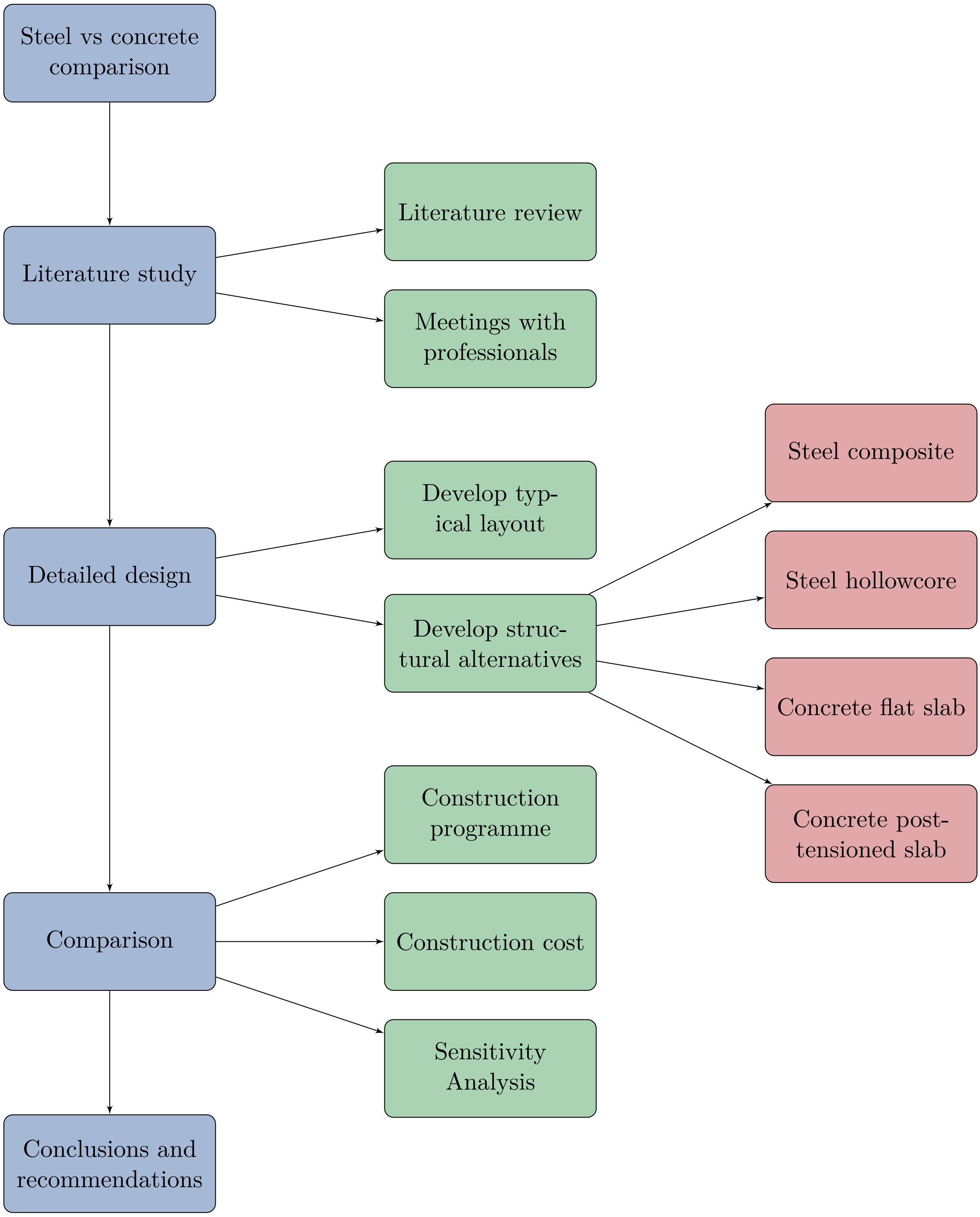


Figure 1.2: Graphical representation of methodology employed in this study

Chapter 2

Literature review and information gathered

Some of the primary objectives of the literature review and background information provided in the study were to:

- Investigate current preferences, both in South Africa and internationally, regarding the use of steel and concrete framing systems for multi-storey office buildings.
- Identify and review existing comparative studies for office buildings in South Africa and internationally.
- Describe the steel and concrete structural alternatives that are typically employed in office buildings in South Africa.
- Identify potential advantages and disadvantages of steel and concrete framed multi-storey office buildings.
- Provide information regarding the fire design of steel framed buildings, which is an important consideration in the cost-competitiveness of steel framed structures.
- Research floor vibrations under human induced loads and the response of steel framed structures. Identify methods to ensure floors vibrations are acceptable from a human comfort point of view.

2.1 Background

Low-rise multi-storey office buildings are required on a regular basis all over the world, and although high-rise office structures might be more striking and iconic, low-rise structures are required more frequently. It is thus important that these structures are constructed as cheaply and efficiently as possible. Diverse preferences exist in different parts of the world with regards to the preferred construction method for these low-rise commercial structures. Furthermore, a construction method that proves to be the best today, may not necessarily be the best method in the future (Lombard, 2011). It is therefore important that, with changing circumstances, all structural alternatives are considered to ensure that the best solution can be implemented during a project.

At present there is little to no available available information which provides a comparison between different structural alternatives when used for the structure of a low-rise multi-storey office building in South Africa. This lack of comparative information can lead to uncertainty among project teams when choosing a structural solution during a project. Mathys (2003) states that the greatest opportunity to achieve value for money is at the inception of a project. It is therefore important that the advantages and disadvantages of different structural alternatives are understood so that the best solution can be identified and chosen at an early stage during a project.

2.2 Current preferences in the multi-storey industry

There are diverse preferences in different parts of the world regarding the selection of the structural framing material for multi-storey office buildings. This section explores some of the current preferences that exist both in South Africa and internationally.

2.2.1 Preferences in the South African construction industry

During the course of this study several meetings were held with professionals in the South African building industry. Discussions were held with professionals encompassing a wide range of disciplines, with some of the occupations including:

- Consulting engineers
- CEO of a steel fabrication company
- Quantity surveyors
- Building contractors
- Fire protection specialists
- Construction project management specialists

Summarised minutes of these meetings were included in Appendix D. Through the meetings and discussions with the professionals, an opinion was formed regarding current perceptions that exist within the South African building industry regarding the use of steel and concrete framing systems for multi-storey office buildings. Some of the major trends and perceptions that were identified included the following:

- **Concrete preferred over steel** - Concrete is by far the dominant framing material for multi-storey office buildings in South Africa, and is the default option when selecting a framing material for these buildings. There is currently no information that could be traced which was able to provide an indication of the market share of steel and concrete framing systems for multi-storey office buildings in South Africa, but it is believed that concrete is the overwhelming favourite. Steel framed multi-storey office structures are very rarely encountered in South Africa and, if employed, will often be dictated by the client's requirements.

- **Concrete considered cheaper** - Concrete structures are considered to be cheaper than their equivalent steel counterparts. The main reason that was often cited for selecting a reinforced concrete frame, as opposed to a steel frame, was the perceived difference in cost, even without any specific knowledge regarding the actual cost difference. In addition, professionals are familiar with concrete design and its implementation. There is a fear that by choosing a steel framed structure, a more challenging construction route may have been chosen. The detailing of concrete structures tends to be relatively simple, and concrete construction allows for additional flexibility during the construction process.

  The cost decision is also frequently made in terms of limited criteria. The influence of time-related costs are rarely considered when evaluating the cost effectiveness of different structural options. This can include costs such as the ability to earn income at an earlier stage, a reduction in interest costs due to a shorter construction programme, and lower preliminary and general (P&G) costs.

- **Early collaboration and involvement of steel fabricator** - A recurring theme encountered during meetings was the importance of early collaboration, and involvement of the steel fabricator in order for steel projects to be successful. Early collaboration enables aspects such as the building layout, cladding systems, steel sections, fire protection materials and other components to be chosen in such a way that they allow for the full benefits of steel construction to be realised.

- **Shift in mindset required for steel to be successful** - There is an understanding that steel construction can provide certain advantages, such as an increased speed of construction compared to concrete construction methods. Many of the participants believelied that there is potential for the increased use of structural steel in multi-storey office buildings, and that it may be competitive with reinforced concrete buildings in the future.

  In order for steel to be more widely used and accepted a change in perception would be required among industry professionals, ranging from engineers to quantity surveyors, building contractors and even clients. One of the biggest challenges to implementing steel in projects is to shift away from the idea of reinforced concrete being the best option.

2.2.2 International preferences

Several studies have been undertaken internationally to highlight preferences that exist with regards to the selection of the framing material for multi-storey buildings.

2.2.2.1 Great Britain

The British Constructional Steelwork Association (BCSA) and Tata Steel have conducted a survey, running over many years, to reveal the market share of structural framing materials in the non-domestic, multi-storey building market in Great Britain (Barrett Byrd Associates, 2016). Figure 2.1 reveals the results of this study.

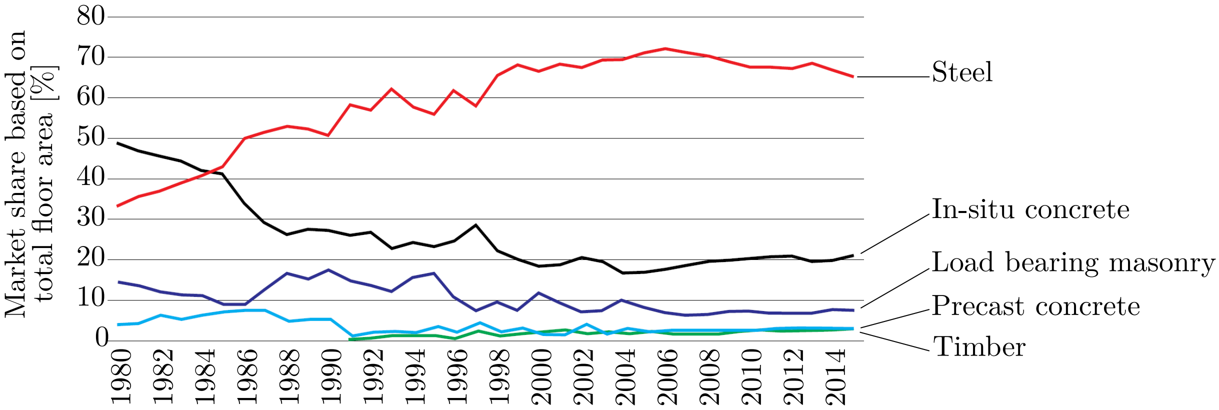


Figure 2.1: Market share of structural framing materials for multi-storey buildings in Great Britain (Barrett Byrd Associates, 2016)

Figure 2.1 illustrates that steel currently holds a dominant share in the multi-storey construction market in Great Britain. This has been the case for the past 15 years, where steel framing systems have frequently attained a market share of approximately 70 %. In 2015, steel frames accounted 68 % of the multi-storey office building market, compared to just 23 % for in situ reinforced concrete (Barrett Byrd Associates, 2016), which includes post-tensioned slabs.

2.2.2.2 USA

The American Institute of Steel Construction (AISC) published a report providing an industry overview regarding the use of structural steel in the United States of America (American Institute of Steel Construction, 2012). One component of this overview showed the market share of structural framing materials for non-residential, and multi-storey residential construction, where "multi-storey" is considered to be greater than 4 stories. Table 2.1 shows market share for various structural framing materials in the USA.

From Table 2.1 it is clear that structural steel is the dominant framing material in the USA for non-residential and multi-storey residential buildings, with a market share of 57.5 % in 2011. This is double the closest competing framing material, reinforced concrete, which held a market share of 20.8 % in 2011.

In some other countries new construction trends are shifting towards the increased use of structural steelwork. India is beginning to shift away from on-site fabrication to workshop delivery models, and it is anticipated that this shift will fuel the demand for steel structures in India (Shah, 2012). Although concrete remains the default construction option in India, steel

Table 2.1: Market share of various structural framing materials for non-residential, and multi-storey residential construction in the USA (American Institute of Steel Construction, 2012)

	Market share of various framing materials [%]							
	2004	2005	2006	2007	2008	2009	2010	2011
Structural steel	51.7	51.9	52.2	53.4	54.8	56.3	58.1	57.5
Reinforced concrete	20.8	22.1	23.1	24.5	21.7	18.3	20.1	20.8
Pre-engineered buildings (Steel)	5.6	5.3	4.9	4.6	5.2	6.2	5.7	5.6
Wood	6.6	6.8	6.9	5.5	5.8	6.1	6.3	7.1
Masonry	7.4	7.5	6.8	6.2	5.9	6.3	5.8	5.3
All Other	7.9	6.5	6.1	5.8	6.6	6.8	4	3.7

fabricators and building developers are beginning to appreciate the advantages of using structural steel as opposed to reinforced concrete. This has resulted in a slow but gradual shift in perception towards the increased use of steel construction methods.

2.3 Comparative studies

No comparative studies between steel and concrete framed multi-storey office buildings in South Africa could be found. Internationally however, a variety of comparisons have been published. Some of the main comparative studies that were investigated during the course of this study are reviewed in this section.

2.3.1 Tata Steel and the British Constructional Steelwork Association

Tata Steel, formerly known as Corus, and the British Constructional Steelwork Association (BCSA) have conducted comparisons between steel and concrete buildings for the past 25 years, with the first comparison study undertaken in 1992 / 1993. The results of the most recent study are presented in this section, with all details obtained from Barrett Byrd Associates (2016). This study was first developed in November 2011 by the BCSA and Tata Steel, and built on previous studies to reflect changes in construction techniques, and the prevalence of different structural framing solutions (British Constructional Steelwork Association and Tata Steel, 2016). Various industry professionals were commissioned by the BCSA and Tata Steel to perform an objective study of current construction practice for multi-storey office construction in Great Britain. The study set out to compare two typical office buildings, across a number of aspects, using a variety of structural solutions. The results of the study provide design teams and quantity surveyors with cost and programme guidance when selecting structural solutions for multi-storey office buildings. The results of the study are updated each year to ensure that it remains relevant and up to date.

2.3.1.1 Methodology

For the purposes of the comparison, two typical office building configurations were used, and for the remainder of this section, will be referred to as Building 1 and Building 2. Building 1 was identified to be representative of a typical speculative low-rise office building, situated in an out-of-town location. It is rectangular, with a gross floor area of 3200 m^2. The building has 3 stories, is not dictated by site constraints and has a floor width of 18 m to provide open plan floor space. Building 2 was identified to represent a typical office building located in a city centre. It is an 8-storey, L-shaped building with a gross floor area of 16 500 m^2.

Building 1 is more in line with the type of structure that was considered in this thesis and, as such, it will be reviewed in more detail. An architect's impression of Building 1 is shown in Figure 2.2 below.

Figure 2.2: Architect's impression of building 1 (Barrett Byrd Associates, 2016)

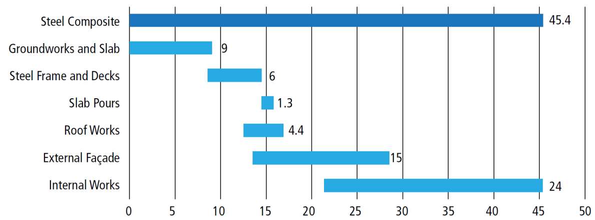
A structural grid of 7.5 x 9 m was established as an efficient layout for a building not dictated by site constraints, and was used for all framing options. The following four structural alternatives were considered, as well as the terms that they will be referred to for the remainder of this section:

1. Steel composite beams and composite slab (Steel composite)
2. Steel frame supporting pre-cast concrete slabs (Steel precast)
3. Reinforced concrete flat slab (RC flat slab)
4. In-situ concrete frame with post-tensioned slab (PT flat slab)

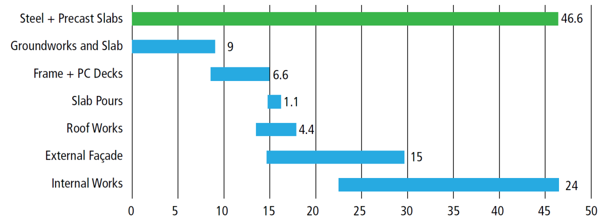
The foundations for each of the options were designed using unreinforced mass concrete pads. Both steel options employ steelwork cross-braced framing for the building's core, with a block-work infill for the building envelope. The concrete structural alternatives make use of concrete shear walls to provide lateral stability, and a 30 minute fire rating was specified for all structural alternatives.

2.3.1.2 Results

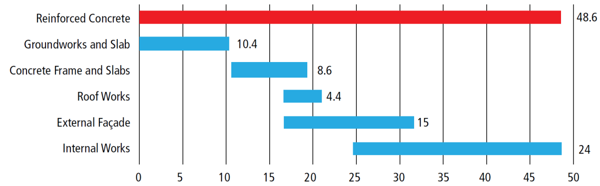
The study recognised that the selection of the framing material is not based solely on cost, and as such the construction programme and buildability implications for each of the options was considered. The construction programme for each of the options is presented in Figure 2.3.



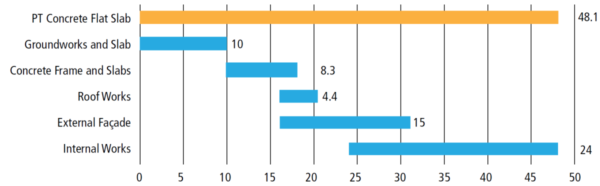
(a) Steel composite



(b) Steel precast



(c) Concrete flat slab



(d) Concrete post-tensioned slab

Figure 2.3: Comparison of the construction programme for each of the steel and concrete structural alternatives [weeks] (Barrett Byrd Associates, 2016)

The comparison of the construction programmes revealed that both steel buildings were able to provide shorter construction programmes than the concrete buildings. The steel composite structure was able to provide the shortest construction period, which was more than 2 weeks shorter than both concrete framed structures.

The impact of the construction programme, together with the logistics and buildability analysis, were reflected in the cost comparison for each of the structural alternatives. Both steel options benefit from lower preliminary and general (P&G) costs due to their shorter construction programmes. The study did not simply consider the cost of the structural frame, but of all elements contributing to the total building cost. Therefore the cost of the foundations and substructure, roof, and external cladding were calculated separately for each structural alternative, as opposed to simply keeping these costs constant across all options. The results of the cost comparison are shown in Table 2.2.

Table 2.2 shows the cost of various building components, in addition to the total building cost. Both steel structures benefited from lower foundation costs, due to their reduced self weight compared to the concrete structures. The RC flat slab structure provided the lowest frame cost of all the structural alternatives, with the steel composite frame cost proving to be marginally more expensive. The steel composite option provided the lowest total building cost, and was found to be almost 10 % cheaper than the cheapest concrete framed structure. The steel structure with precast units was cheaper than the both of the concrete alternatives, but more expensive than the steel composite structure by approximately 6 %.

Table 2.2: Table of building costs for Building 1 [£/ m^2] (Barrett Byrd Associates, 2016)

City of London costs	Steel composite	Steel precast	RC flat slab	PT flat slab
Substructure	70	74	90	84
Frame and upper floors	174	193	172	202
Frame cost % of total cost	8.96%	9.39%	8.04%	9.52%
Total building	1941	2055	2138	2121
Percentage difference	0%	5.87%	10.15%	9.27%

2.3.1.3 Conclusions

The study illustrates that, in the UK, for a typical out-of-town office building, steel framed structures were able to offer advantages over concrete framed structures in terms of both time and cost. Furthermore, the study revealed the importance of considering all aspects influencing the total building cost. Although the frame cost of the RC flat slab and steel composite building were found to be similar, once all aspects influencing the total building cost were considered the steel composite structure was shown to be approximately 10 % cheaper than the RC flat slab structure. The reduced P&G costs, substructure and other building costs significantly influenced the total building cost, and cost comparison between different structural options. The frame cost only contributed approximately 10 % of the total building cost. The study therefore revealed, amongst other things, the importance of considering all aspects when performing cost comparisons.

2.3.2 Comparative study undertaken by The Concrete Centre

The second comparative study that was considered, which also took place in Great Britain, was a study undertaken by The Concrete Centre in 2008 (The Concrete Centre, 2008a).

2.3.2.1 Methodology

The methodology adopted in this study was similar to the study conducted by the BCSA and Tata Steel, with the purpose of the study also being to compare the influence that a change in structural solution would have on the cost effectiveness of a multi-storey office building. Leading practitioners in various fields were commissioned in order to ensure that the study would be impartial and not biased in any way. Two layouts, based on structural grids commonly used in Great Britain, were developed and designed using a number of steel and concrete framed building alternatives. The first layout, Building A, was chosen to reflect a typical office building in an out-of-town location while the second layout, Building B, was chosen to reflect a city centre office building. Building A is more in line with the type of building that was considered in this thesis, and it is therefore discussed in more detail. The layout of Building A is shown in Figure 2.4:

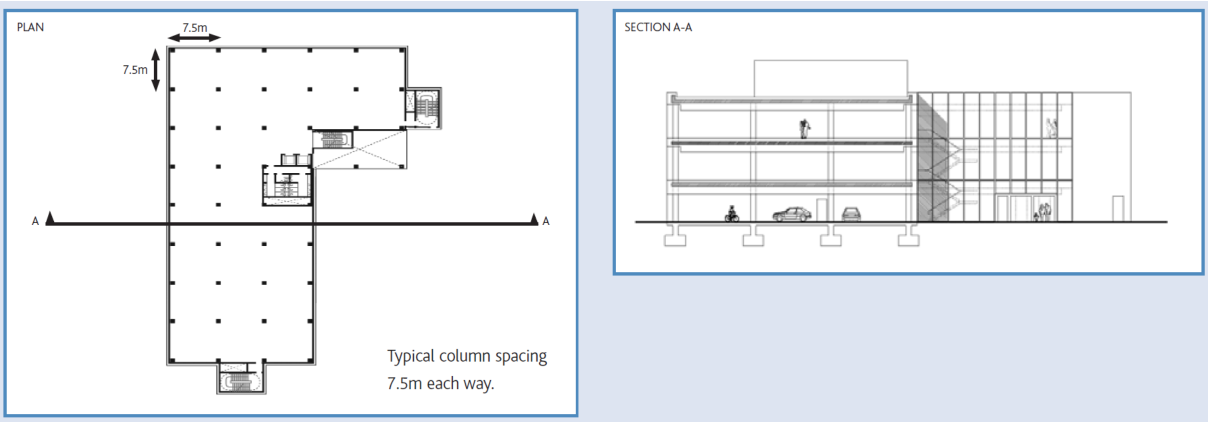


Figure 2.4: Floor plan and section for Building A (The Concrete Centre, 2008a)

Building A is an L-shaped building with a 7.5 x 7.5 m grid spanning in both directions, which is employed in all frame options. It can be seen as being representative of a building of average size situated in a commercial or business park. The building consists of three stories, which result in a gross floor area of 4650 m^2. The building contains one central service core, a full height atrium, a fan coil air-conditioning system and makes use of a curtain walling cladding system.

2.3.2.2 Results

The study compared the overall construction time for each of the structural alternatives, in addition to the time required for various activities, ranging from procurement to completion. All pricing rates were based on recently tendered projects in the South-East of England, which tended to be lower than prices for city-centre projects. A comparison of the construction programme for the building alternatives is shown in Table 2.3.

Table 2.3: Comparison of overall programme times, indicating periods from procurement to completion[weeks] (The Concrete Centre, 2008a)

	Steel composite	Steel hollowcore	Slimdek	RC flat slab	PT flat slab	In-situ hollowcore
Frame procurement time	10	10	10	10	10	10
Frame lead-in time	12	12	12	4	4	4
Frame construction time	8	7	7	10	11	13
Total construction time	48	48	48	50	51	52
Total project time	70	70	70	64	65	66

The results of the construction programme revealed that the concrete framed options were able to provide a shorter total project duration, compared to the steel framed options. Although the frame construction time for the steel options was shown to be significantly faster than the time required for the concrete options, the lead-in time required was three times as long, which led to a longer overall project duration.

The cost implications associated with duration of the construction programme were considered when calculating the construction cost of the various structural alternatives, which is presented in Table 2.4.

Table 2.4: Building costs for various steel and concrete structural options for Building A[£/ m^2] (The Concrete Centre, 2008a)

	Steel composite	Steel hollowcore	Slimdek	RC flat slab	PT flat slab	In-situ hollowcore
Foundations and substructure	41	42	41	43	43	44
Frame and upper floors	122	139	188	122	138	127
Frame cost as % of total cost	8.31 %	9.30 %	12.25 %	8.36 %	9.34 %	8.60 %
Total building cost	1468	1495	1534	1460	1477	1477
Percentage difference	0.55 %	2.40 %	5.07 %	0 %	1.16 %	1.16 %

Table 2.4 showed the cost of various building components as well as the total building cost. The steel structural alternatives benefit from reduced foundation costs due to their lower self weight compared to the concrete framed structures, but this only resulted in a small cost saving. The steel composite and RC flat slab options provided the lowest frame and upper floor cost, however the RC flat slab option offered the lowest total building cost.

2.3.2.3 Conclusions

Overall, the study showed that in Great Britain, both steel and concrete structural alternatives are able to provide a competitive solution when used for a typical multi-storey office structure. Once all factors influencing the building cost were considered, the RC flat slab alternative provided the lowest total building cost by 0.55 % compared to the steel composite structure, which equated to a cost difference of £ 37 825. The concrete framed structures all offered time savings in the total project time, compared to the steel framed structures. This can largely be attributed to the long lead-in time that was required for the steel framed structures.

The total project time was calculated by summing the frame procurement time, the lead-in time and the total construction time. However, this does not provide a very realistic indication of the construction time of the buildings, due to the fact that it does not consider the time taken to construct the building substructure. At the outset of the project, the construction of the building frame cannot commence immediately, and the building foundations and substructure will first need to be constructed. This time period can be used to order the steel components, which assists in reducing the waiting period for the steel to arrive on site. Therefore, by considering the amount of time required for the construction of the foundations and subtracting it from the lead-in time, a more accurate reflection of the total construction time would have been achieved.

2.3.3 Australian Steel Institute

In 2007, the Australian Steel Institute (ASI) commissioned the quantity surveying firm, Rider Levett Bucknall, to develop a comparative cost model study for a steel and concrete framed medium-rise commercial building. The building used for the cost model was a four storey commercial building with one basement level, and was chosen to represent a typical medium-rise commercial building configuration. The objective of the study was to develop a cost model that would demonstrate the relative cost of a steel framed building versus a post-tensioned concrete building. Following on from the original study in 2007, the costs were updated in 2008, 2009 and 2011 to keep the model up to date. The results of the 2011 cost comparison are presented in Figure 2.5.

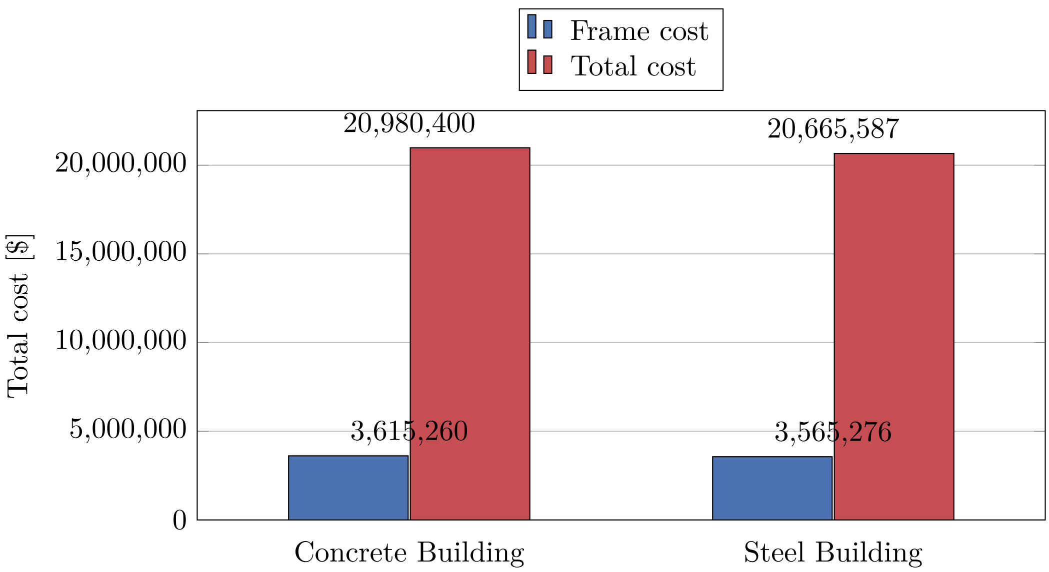


Figure 2.5: Cost comparison of steel and concrete framed medium rise commercial building in Australia (Rider Levett Bucknall, 2011)

The study revealed that the steel framed building was highly competitive with the concrete framed building, in terms of both frame and total project cost, with the steel framed building proving to be 1.5 % cheaper in terms of total project cost. The 1.5 % difference equated to a cost difference of approximately $ 315 000. The study considered the difference in P&G costs due to the shorter programme offered by the steel building, but did not take into account other time related costs in the comparison.

As a result of the cost comparison study, the ASI recognised the potential for the increased use of structural steel, in the multi-storey commercial building market. In order to further explore this potential, the ASI published a report on the cost of structural steel when used for medium rise commercial buildings (Rider Levett Bucknall, 2011). Several medium-to-large sized steel fabrication companies participated in the study, to provide realistic information regarding the cost of steel when used for medium-rise commercial structures. The companies were experienced in terms of having recently tendered on structural steel construction projects, and were familiar with current market conditions (Rider Levett Bucknall, 2011). A significant body of information regarding the actual steel costs for recently tendered projects, current market conditions, the nature of steel framed structures, the pricing of steel, and many other aspects was obtained.

Some of the key findings from market feedback were the following:

- Steel costs need to be developed from first principles, and to simply employ an average tonnage cost is an over simplification. In order to develop accurate steel costs a number of factors need to be taken into account including the supply, fabrication, erection, and any coatings that are required.
- Misconceptions exist regarding how a change in the material cost of steel influences the total cost rate. The material cost only forms part of the overall steel cost, with other factors such as fabrication and erection, also having a significant influence.
- The feedback revealed a limited awareness among industry professionals regarding the cost of steel when used for medium-rise commercial structures. In addition to this, the feedback revealed a lack of suitable medium-rise steel framed buildings that could be used for cost benchmarking analysis (Rider Levett Bucknall, 2011).
- Preconceptions exist among industry builders, engineers, and cost consultants on the cost of steel, which may not be representative of current market conditions. In some cases Australian quantity surveyor's perceptions on steel cost were overestimated by up to 80 %, and this perception was a primary factor in decisions against using steel framing options (Australian Steel Institute, 2012).

The study revealed that actual steel rates achieved for medium-rise multi storey buildings could be significantly lower than for other common uses of structural steel, such as portal frame construction and roof framing. A primary reason for these lower costs can be attributed to the simplicity of the design, and repetitive nature of typical medium-rise commercial buildings. These characteristics lead to savings in connection costs, faster erection times through the use of bolted connections, and the reduction in shop drawings, due to repetition of member types. The report was thus able to conclude that the nature of a typical medium-rise commercial building leads to lower steel costs than for other common steel buildings.

2.3.4 Canadian Institute of Steel Construction

The final comparative study that was reviewed, is a cost comparison study developed by the Canadian Institute of Steel Construction (CISC), which evaluates the impacts of using a steel and concrete framing system for a typical commercial building in Canada. Leading practitioners were commissioned by the CISC to ensure that the study was not biased. The structure used for the comparison was a 6 storey office building, which includes a penthouse and a basement parking level. The building has a gross floor area of approximately 11 100 m^2 and was designed using a steel composite framing system and an in-situ reinforced concrete framing system. The results presented in this section are from the cost study, and can be found in Canadian Institute of Steel Construction (2015). The results of the cost comparison are shown in Table 2.5.

Table 2.5: Cost of building components in \$/m^2 of gross floor area (Canadian Institute of Steel Construction, 2015)

	Steel building	Concrete building	Percentage difference
Foundation	\$ 16.70	\$ 35.69	114%
Frame	\$ 251.50	\$ 318.25	27%
Foundation and frame	\$ 268.20	\$ 353.94	32%

The costs presented in Table 2.5 only include the costs associated with the building's foundation and frame, and do not consider the cost of non-structural components, such as cladding, mechanical and electrical services, finishes and fitments, to name a few. The steel structure benefits from a significantly lower foundation cost due to its reduced self weight, with the foundation cost of the steel structure proving to be less than half of those for the concrete structure. The cost of the steel building's frame was also lower than the concrete building, and was found to be 27 % cheaper. The total cost of the foundations and frame for the steel framed building were 32 % lower than for the concrete framed building. There are additional cost benefits that can be associated with the steel frame building such as a faster speed of construction, a reduction in preliminary and general costs, and the ability to build more easily during winter months, however these were not considered in the cost comparison.

In addition to the cost comparison, the study included a whole building life cycle analysis on the sustainability impacts of the steel and concrete frame buildings. The life cycle analysis was conducted by Ryerson University and the results are presented in Figure 2.6. A variety of environmental impact categories were considered, and considering Figure 2.6 it is clear that the steel framed structure provides a lower environmental impact in the majority of the categories that were considered.

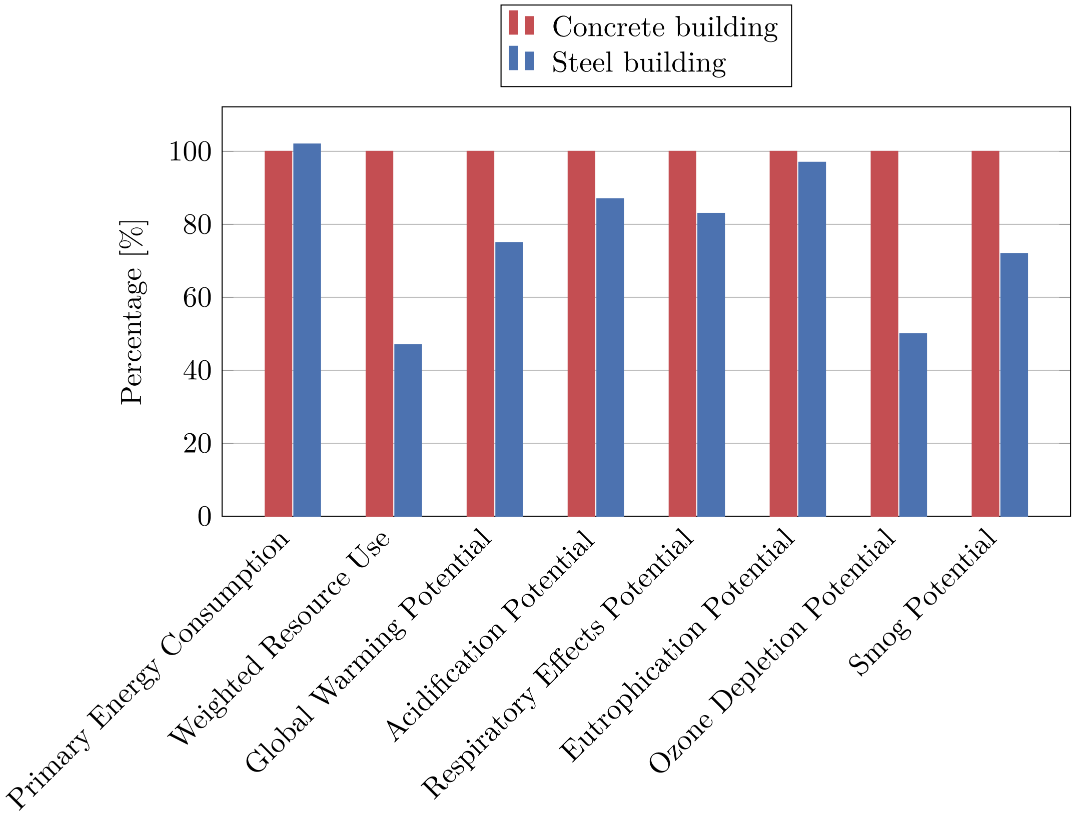


Figure 2.6: Life cycle analysis of sustainability impacts of steel and concrete framed buildings (Canadian Institute of Steel Construction, 2015)

2.4 Floor systems for multi-storey office buildings

This section reviews the literature regarding various floor systems that can be used in steel and concrete framed multi-storey office buildings. These floor systems were identified through research into solutions that are frequently used in multi-storey office buildings.

2.4.1 Composite steel-concrete floor systems

Composite construction currently dominates the non-residential multi-storey building sector in the United Kingdom (Steel Construction Institute, 2015), and is gaining popularity in South Africa. Its success can be attributed to the strength and stiffness that can be achieved with minimum use of materials (Steel Construction Institute, 2015). The use composite construction results in a highly efficient and lightweight design. This can be attributed to the fact that concrete is strong in compression while steel performs well in tension. Through the combination of the favourable properties of steel and concrete, floor systems that are both safe and economic can be developed (Vasdravellis et al., 2012).

2.4.1.1 Composite beam and slab with profiled steel sheeting

Composite metal deck floors typically consist of profiled steel sheeting supporting a reinforced concrete slab. The steel decking not only serves as permanent formwork to the concrete slab, but once the concrete has gained sufficient strength, allows the concrete slab to span between the supporting steel beams. Steel sections act compositely with the concrete slab through the addition of headed shear studs which are most often welded through the steel deck onto the top flange of the beam. A cross section of a typical composite steel deck floor is shown in Figure 2.7.

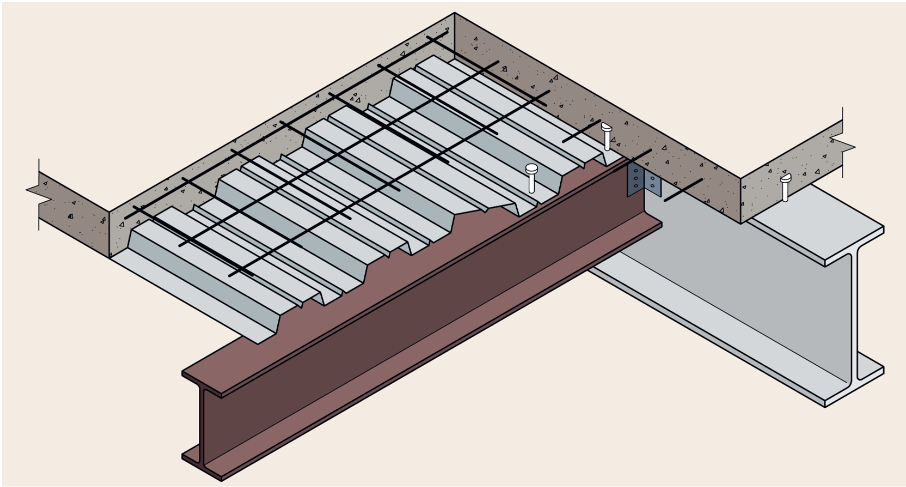


Figure 2.7: Cross section of composite steel deck floor (Simms and Hughes, 2011)

Composite construction with metal deck flooring provides a number of advantages. Some of the main advantages are summarised below:

- **Speed of construction** - The use of this floor system enables rapid construction speeds to be achieved, particularly when unpropped construction is employed. Many sheets of profiled steel sheeting can be bundled together and moved in a single lift. This enables greater ease of transportation and handling on site. Decking can be lifted in bundles and spread by hand across the floor area to where it is required. This serves to greatly reduce the number of crane lifts required, compared to precast concrete construction. In excess of 400 m^2 of metal decking can be placed by one team per day (Rackham et al., 2009). Additionally, minimal reinforcement is required due to the tensile reinforcement provided by the steel deck. Large areas of floor can be poured quickly, and floors can be concreted in rapid succession (Rackham et al., 2009).

- **Early access for following trades** - Once erected, the metal decking is able to immediately provide a safe working platform, and following trades can operate on the floor immediately below the one being cast (Wright et al., 1987).

- **Low self-weight** - Typical self weights vary from 2 - 3.5 kN/m^2 which is between 40 - 60 % of the weight for a typical reinforced concrete flat slab floor system (Davison, 2012). This lower self weight leads to smaller foundations being required, which can offer cost and time benefits.

- **Transport savings** - Steel sheeting can be easily stacked, transported and handled, and one truck can transport in excess of 1000 m^2 of decking. This greatly reduces the number of deliveries required to site compared to other construction methods (Rackham et al., 2009).

A fundamental decision that has to be made when designing composite steel beams is whether propped or unpropped construction will be employed. A description of both of these construction methods, as well as the design implications associated with each method, is discussed below:

- **Unpropped construction** - As the name suggests, composite floors constructed using unpropped construction receive no propping during the construction period. Two major advantages of using this method is that it allows for rapid construction speeds to be achieved, and it allows early access for following trades (South African Institute of Steel Construction, 2013). Composite beams are usually unpropped to ensure that these advantages are realised (Davison, 2012). One drawback of using unpropped construction is that the steel beam is required to support the weight of the fresh concrete prior to composite action being attained. Another disadvantage, particularly in long span floor systems, is that the deck sags under the weight of the fresh concrete which can lead to "ponding" of concrete, resulting in additional concrete weight.

  The decision to make use of unpropped construction influences both the bending resistance checks that are required, as well as the final deflection of the composite beam. The bending resistance of the composite beam needs to be checked in both the construction and composite conditions. During the construction condition, the concrete has not gained sufficient strength for composite behaviour to occur so the bending resistance is that of the steel beam alone. Additionally, the final deflection of the beam is influenced. Due to the absence of any props the deflection under the weight of the fresh concrete is calculated using the properties of the steel section alone. Deflection is often a critical consideration in unpropped construction and larger beam sizes may be required to meet serviceability criteria. The pre-cambering of beams can also be used to reduce deflection where required. According to Rackham et al. (2009) propping should be avoided wherever possible as it reduces the speed of construction, which therefore affects the construction sequence and economy of the building.

- **Propped construction** - Propped construction eliminates some of the main drawbacks associated with unpropped construction. Props are left in place until the concrete has gained sufficient strength to act compositely with the steel beam. This means the moment resistance and deflection need only be checked in the composite condition. There are two major disadvantages associated with propped construction. Firstly, the speed of construction is reduced due to having to erect and remove props. Secondly, the presence of the props inhibits the ability to begin with following trades. An example of temporary props is shown in Figure 2.8 below:

Figure 2.8: Example of temporary propping of a composite steel deck floor (Rackham et al., 2009)

2.4.1.2 Steel framed building with precast hollowcore floor units

The second steel-concrete floor system that is considered is a steel framed building supporting precast concrete hollowcore floor slabs. Hollowcore units were originally conceived and developed some 25 years ago, as an alternative to in-situ concrete floors for multi-storey construction (Concrete Manufacturers Association, 2011). Hollowcore floor slabs are precast concrete floor slabs with hollow cores running longitudinally through the slab. They are prestressed, which enables them to span further than normal reinforced concrete. They are typically manufactured in standard widths of 1.2 m, and can be constructed with varying thicknesses and reinforcing configurations, to suit different spanning requirements. Hollow-core floor units are popular for a wide range of uses with applications ranging from suspended flooring and security walls, to retaining walls and reservoirs. Due to their popularity hollowcore units are available in most parts of South Africa (Concrete Manufacturers Association, 2011). An example of a cross section of a 200 mm deep hollowcore unit is shown in Figure 2.9:

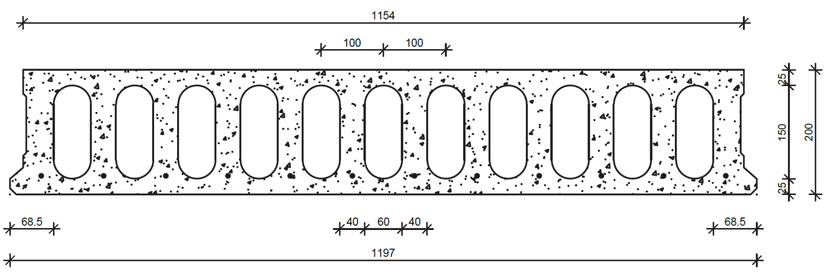


Figure 2.9: Cross section of hollow-core unit (Concrete Manufacturers Association, 2011)

Using hollowcore floor units supported by steel members offers a number of advantages over in-situ concrete construction methods. Some of the main advantages are the following:

- **Speed of erection** - Hollowcore units are manufactured off-site and arrive to site ready to be erected. They require no propping during construction, and for indoor applications only a 40 mm levelling screed is required. Erection rates of up to 600 m^2/day of floor area are possible (Concrete Manufacturers Association, 1999).

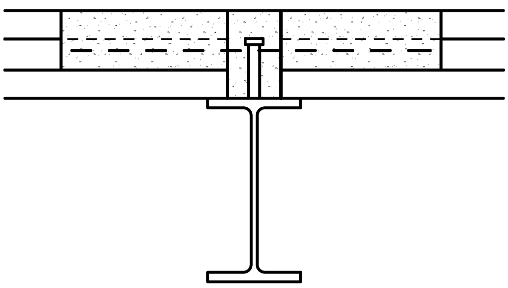
- **Self weight** - Hollowcore units offer a reduction in self weight of up to 30 % compared to a reinforced concrete slab of the same depth. This reduction in self weight subsequently reduces the size and cost of the foundations that are required.

- **Quality control** - Hollowcore units are manufactured off-site in a controlled environment, which allows protection from the weather and enables manufacturing to occur under all weather conditions. Steel and precast hollowcore units come from a manufacturing technology rather than a site based activity, and as such they share the quality control, accuracy and reliability of factory production (Hicks and Lawson, 2003).

An important decision that has to be made when designing steel beams supporting hollowcore slabs, is whether or not composite construction will be used. Composite construction with precast hollowcore slabs is currently one of the most common construction methods in the United Kingdom (Lam, 2007).

Composite design of the steel beam and hollowcore slab utilizes the favourable properties of steel and concrete to develop a more efficient structural solution than if these materials were designed to act in isolation. Both the bending strength, as well as the stiffness, is increased at little additional cost, except for the welded shear studs (Lam, 2007). The design of composite steel beams supporting hollow-core units is similar to that of composite steel beams with metal decking, in that both the construction and composite condition must be considered.

South African design codes do not deal explicitly with the design of composite beams supporting hollowcore units. This subject is covered extensively in Design Guide Publication 287 (Hicks and Lawson, 2003) of The Steel Construction Institute. Publication 287 provides design guidance of composite beams using precast concrete slabs, and was used for the design of the composite beams in this study. An example of a typical composite connection between a steel beam and hollowcore slab is shown in Figure 2.10.



(a) Composite Hollowcore floor section

(b) Composite floor configuration prior to screeding

Figure 2.10: Composite floor construction with precast hollowcore units and steel beams (Hicks and Lawson, 2003)

Composite action between the steel beam and hollowcore slab can be achieved through relatively simple detailing requirements. Shear studs are welded to the top flange of the steel beam in order to structurally tie the beam and slab together. Reinforcement is also required, which runs perpendicularly to the steel beam, into the hollow-core slab. This involves trimming off a portion of each slab end and placing steel reinforcement into the hollow cores. Once the screed has been poured and allowed to harden, composite action can now occur. A limit is placed on the minimum width of the flange to ensure sufficient bearing, and to allow space around the shear connector for the grout to penetrate. According to the Concrete Manufacturers Association (2011) the width of the flange must be at least 171 mm in order for composite action to be attained, and to allow sufficient bearing on the steel beams.

Composite design adds some complexity to the design and construction of the steel beam and hollowcore floor system, but it can result in significant cost savings. Minor savings are possible in amount of prestressing cables required, due to the fact that continuity now exists over the supports. However, the main area where cost savings can be achieved is with regards to the size of the steel beams that are required. The reduction in the section size, and mass of the steel beams that are required will be discussed in more detail in Section 3.7.

2.4.2 Reinforced concrete flat slab

Reinforced concrete flat slab structures are versatile and frequently used in multi-storey construction. Markets where flat slab structures are frequently employed include residential, commercial and hospital buildings, to name but a few (The Concrete Centre, 2016). Flat slabs are popular for office buildings as they provide fast construction speeds, allow easy service distribution due to the flat soffit, and the absence of beams allows for reduced floor-to-floor heights (Goodchild, 1997).

Due to the absence of floor beams, punching shear around column heads can often be a critical consideration. The fact that the floors are not prestressed means that flat slabs only possess a limited spanning capability with typical spans ranging between 4 and 10 m (The Concrete Centre, 2016). An example of a flat slat floor system is shown in Figure 2.11.

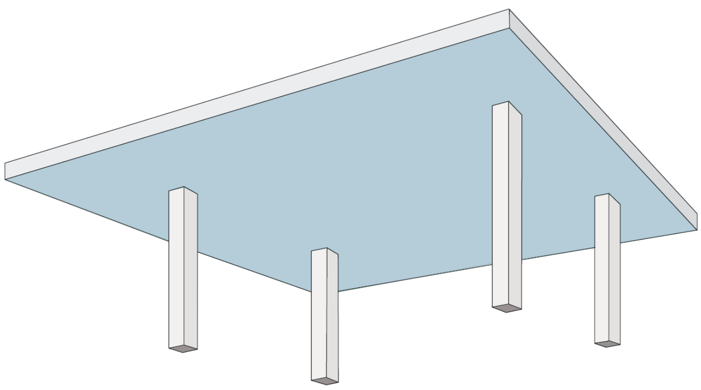


Figure 2.11: Example of a flat slab floor system

2.4.3 Post-tensioned concrete slabs

Post-tensioned floor construction has gained wide acceptance and popularity in many parts of the world. These floors are frequently used in multi-storey construction in North America, Australia, the United Kingdom and the Middle East, while in California post-tensioning is the primary choice for concrete floors (The Concrete Centre, 2008b). Post-tensioned floors are a popular construction method in South Africa, and have been shown to provide cost effective floor solutions.

The construction of post-tensioned floors involves pouring concrete around carefully installed, unstressed tendons in sleeves. Once the concrete has gained sufficient strength, the high strength tendons are tensioned, and locked off at anchor positions. The tensioning of the high strength tendons results in a compressive stress being induced in the concrete slab. The pre-compression of the concrete floor allows the concrete to remain in compression when flexing under applied loads, and this results in a more efficient design when compared to traditional reinforced concrete construction methods (The Concrete Centre, 2008b). The deflection of the slab is also counteracted. Post-tensioned floors can be constructed using either bonded or unbonded construction.

Some of the main benefits of using post-tensioned floors over reinforced concrete floors are the following:

- Increased spanning ability.
- Reduced deflections and slab thickness.
- Lower tendency to crack, which is of particular value for parking decks and liquid retaining structures.

In addition to these benefits, post-tensioned concrete floors are compatible with fast construction speeds, and are relatively easy to design and construct (Goodchild, 1997).

2.5 Benefits of steel construction in multi-storey buildings

The use of steel construction in multi-storey buildings allows for a number of advantages to be realised, with some of the main advantages including:

- **Speed of construction**: Steel construction makes use of pre-fabricated components which can be rapidly assembled on site, thus leading to significant reductions in construction time compared to other construction methods. The construction of the primary frame and floors of a steel framed building can be up to 40 % faster than for a reinforced concrete framed building (Steel Alliance, 2010). Shorter construction programmes lead to a number of time-related savings, such as a the ability to earn income at an earlier stage, lower interest charges on borrowed capital and savings in P&G costs. Furthermore, for inner city projects, reducing disruption and disturbance to nearby roads and buildings can be an important consideration. Steel construction enables a dramatic reduction in the impact of the construction operation on the surrounding buildings and roads (Davison, 2012).

- **Early access for follow-on trades**: The pre-fabricated nature of steel construction leads to a substantial reduction in site activities and allows the installation of follow-on trades to begin at an early stage. Steel construction techniques require significantly less propping to floor systems, if propping is required at all, which enables early access for following trades, such as installation of mechanical and electrical services, cladding and finishes.

- **Reduction in self-weight**: Steel construction results in buildings that are lightweight, even when concrete floors are used, and employ an efficient use of resources. The total building weight for a steel framed building can be up to 30 % lighter than the equivalent concrete building (Steel Alliance, 2010). This reduction in self-weight results in reduced foundation sizes and costs.

- **Quality and safety**: Steel components are manufactured, and fabricated off-site under controlled conditions, which leads to improved quality control. The use of pre-fabricated components allows for a reduction in site activity of up to 75 % for the frame construction compared to concrete construction methods (Davison, 2012). This reduction in site activity results in far fewer construction workers being required on site, and an increased overall construction safety.

- **Long spans and service integration**: Long spans can be realised using steel construction, and spans of 12 - 18 m are frequently achieved using various structural steel technologies (Steel Alliance, 2010). Long span buildings are appealing to building owners and occupants because they allow greater flexibility in current, and future use, due to column-free floor space. Additionally, services can be integrated within the depth of the long span beams, thus reducing floor-to-floor height by up to 300 mm per floor (Steel Alliance, 2010). This reduction in floor-to-floor height can also result in a significant reduction in cladding costs, particularly as the number of stories increases. A 300 mm reduction in floor-to-floor height can result in a saving of between 20 - 30 Euros per square metre of floor area (Davison, 2012).

- **Sustainability benefits**: Steel is one of the most recovered and recycled materials, and approximately 95 % of structural steel sections are recycled in Great Britain (Steel Alliance, 2010). Of the 95 % that is recycled, approximately 10 % is simply reused. The speed of construction and reduced disruption of the site also imply local environmental benefits (Davison, 2012).

2.6 Benefits of concrete construction

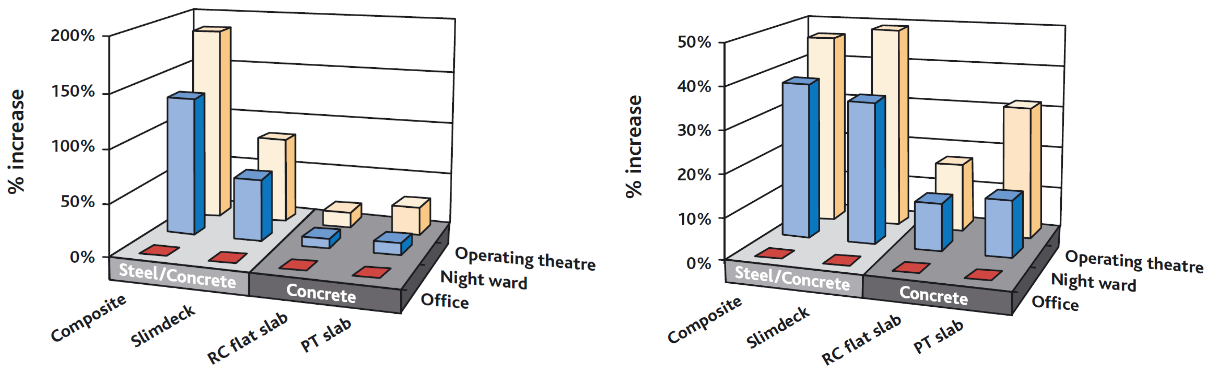
As in the case of steel construction, the use of concrete construction for multi-storey buildings has a number advantages associated with it. Some of the primary advantages are the following:

- **Robustness and durability**: Concrete is, by its nature, a very robust and durable construction material. It is capable of withstanding explosions, accidental damage and vandalism and requires very little, to no maintenance (The Concrete Centre, 2016). Correctly designed concrete requires no additional coatings to protect it against deterioration.

- **Lead in time**: Concrete construction is a common method of construction and, as a result, the lead-in time for concrete framed buildings is typically very short. The lead in time for a flat slab concrete structure is approximately 4 weeks, which will be shorter than typical lead in times for steel buildings (The Concrete Centre, 2006). The primary materials that are required for concrete construction, namely cement, water, sand and stone, are all readily available materials that can be obtained on short notice. This contributes to the short lead-in time for concrete construction.

- **Fire resistance**: Concrete structures have an inherent fire resistance and often do not require any additional fire protection. This can eliminate the cost of having to provide fire protection, which can typically account for around 10 - 15 % of the frame cost in steel framed buildings (Barrett Byrd Associates, 2016). Furthermore it eliminates the need for an additional trade on site, which can assist in reducing construction time.

- **Vibration control**: For the majority of concrete framed buildings, floor vibration criteria are satisfied with no alterations to normal design procedures. Furthermore, for building uses with particularly strict vibration criteria, such as hospitals or laboratories, concrete framed buildings have been shown to be suitable with only minor changes being required. This is in contrast to other building materials where the floor mass or thickness may have to be significantly increased in order to meet vibration criteria (The Concrete Centre, 2016). Figure 2.12 shows a study that was performed into the vibration performance of various steel and concrete floor systems when designed as hospital floors.



(a) Required increase in floor mass

(b) Required increase in floor thickness

Figure 2.12: Increase in mass and thickness for various floor systems and uses (The Concrete Centre, 2006)

Figure 2.12 shows that only relatively minor changes were required to the concrete floor systems to allow them to satisfy even very strict vibration criteria. This is in contrast to steel framed floor systems, where large increases in both the floor thickness and mass may be required to satisfy strict vibration criteria. The fact that concrete floors were able to satisfy vibration criteria with only minor changes, provides concrete buildings with flexibility for change in building use.

2.7 Ultimate loading and Serviceability criteria to be met by structures

The criteria relating to the resistance of structures under ultimate and serviceability loads are well known and covered in South African design codes, (*The Structural Use of Concrete, Part 1: Design* 2000) (*The structural use of steel, Part 1: Limit-states design of hot-rolled steelwork* 2011), and will not be discussed further in this section. However, the fire resistance and floor vibration, particularly of steel framed buildings, are areas which are not comprehensively covered in these design codes. This section provides guidance on the design of steel framed buildings to ensure that they satisfy fire resistance requirements, and that floor vibrations are of an acceptable level.

2.7.1 Fire resistance of steel framed buildings

Fire engineering is an important design consideration, particularly for steel framed buildings. The reason why it is important is because steel loses its strength at elevated temperatures. Figure 2.13 illustrates the yield strength of steel as the temperature increases. From Figure 2.13 it can be seen that between 400 °C and 600 °C, the yield strength factor reduces from 1.0 to 0.47, and by 800 °C steel has only 11 % of its strength at ambient temperatures. It is therefore important to limit the temperature of steel to ensure that is has sufficient strength during fire loading conditions.

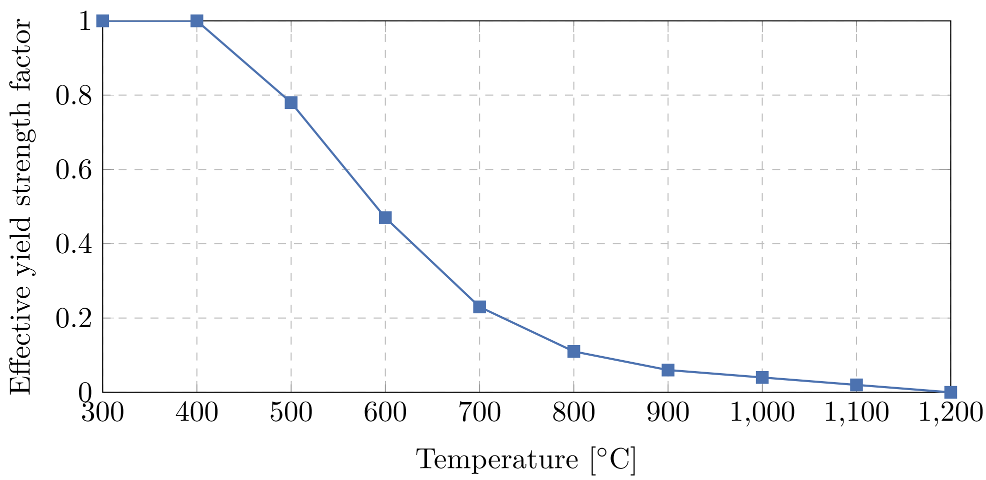


Figure 2.13: Reduction in yield strength of normal structural steel at elevated temperatures (Association for Specialist Fire Protection, 2014)

As mentioned in Section 2.6, concrete structures have an inherent fire resistance, and therefore frequently require no additional fire protection (The Concrete Centre, 2016). SANS 10100 (*The Structural Use of Concrete, Part 1: Design* 2000) specifies minimum values for concrete cover, and minimum dimensions for different building components in order to meet various fire ratings. These requirements can typically be met with relative ease.

The study therefore focuses on the fire design of steel framed members, where fire resistance requirements are often more difficult, and costly, to satisfy. Some important considerations relating to the fire design of steel framed structures are discussed in Sections 2.7.2 to 2.7.4.

2.7.2 Fire rating and temperature development

Fire rating requirements are typically measured according to the length of time a structure can withstand a standard fire. This duration is normally specified in minutes, and is expressed using ratings of either 30, 60 or 120 minutes. The South African National Building Regulations (SANS 10400), Part T - Table 5 specifies fire rating requirements for different building occupancies, and number of stories (*The application of the National Building Regulations* 2008). An excerpt from this table is shown in Table 2.6, and indicates the fire rating requirements for an office building. Table 2.6 shows that for an office building with 3 - 10 stories, a 60 minute fire rating is required. This is accepted as the required fire rating for the office building considered in this study.

Table 2.6: Fire resistance requirements for office buildings according to SANS 10400 - Part T: Table 5 (*The application of the National Building Regulations* 2008)

Type of occupancy	Class of occupancy	Stability [minutes]				
		Single-storey building	Double-storey building	3 - 10 storey building	11 storeys or more	Basement in any building
Office	G1	30	30	60	120	120

As mentioned previously a 60 minute fire rating means that a building can withstand a standard fire for 60 minutes. It is thus important to know what a standard fire means and what temperature can be expected to be reached during such a fire. A standard fire was developed in 1918, and represents a worst case time-temperature relationship between a fire and a structure (Walls, 2015). The temperature increases steadily with time and does consider a cooling phase. Additionally, factors such as the actual fire load, ventilation characteristics, and building properties, which have been shown to affect fire behaviour, are not considered. It can be suitable for short duration fires, but for medium to long duration fires it can become over conservative.

Although a standard fire does not necessarily represent the exact temperature that would be experienced during a real fire, it is still valuable because it allows the performance of different members and designs to be compared in a consistent manner. Additionally, fire ratings of materials and elements are often supplied in terms of a standard fire, so it remains important.

The temperature growth of a standard fire, as well as the temperature of unprotected steelwork in such a fire is shown in Figure 2.14. From Figure 2.14 it can be seen that after one hour of a standard fire, the temperature of unprotected steel would be approximately 950 °C. Figure 2.13 revealed that the yield strength of steel at this temperature would be only 6 % of the yield strength at ambient temperature. It is therefore clear that the temperature of the steelwork needs to be limited to ensure that it retains sufficient strength during the fire loading condition.

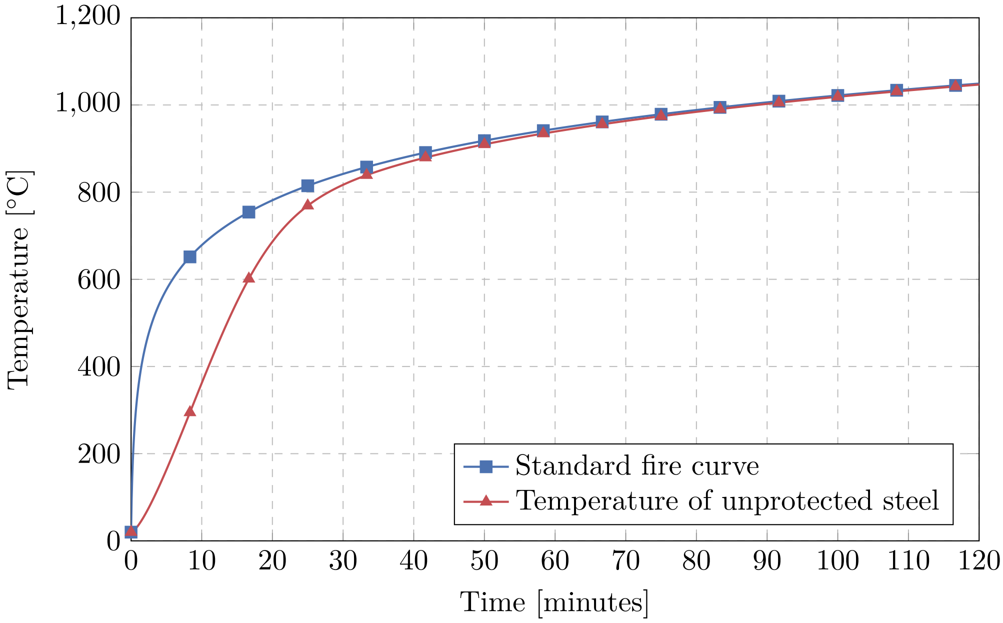


Figure 2.14: Temperature development of standard fire curve and unprotected steelwork

2.7.3 Steel member design for fire

It has been established that for a 1 hour fire rating the temperature of steel members will need to be limited to ensure that they retain sufficient strength at the fire limit state. This section provides information regarding the design of steel members to ensure that they do not fail during the fire loading condition.

2.7.3.1 Limiting and critical temperatures

In order to ensure that the steel retains sufficient strength at the fire limit state the Eurocode recommends the following limiting temperatures when designing various steel sections for different building occupancies (Association for Specialist Fire Protection, 2014). The critical temperatures are shown in Table 2.7.

Table 2.7: Default critical temperature from the Eurocodes [°C] (Association for Specialist Fire Protection, 2014)

Building occupancy	Non-composite beams supporting concrete floor slabs	Composite beams supporting concrete floor slabs	Hot rolled H-section columns in compression	Hot finished / formed structural hollow sections
Office / domestic	603	576	563	572
Storage	576	544	530	512
Shopping / congregational	583	553	539	521

2.7.3.2 Passive protection to steel members

Passive fire protection is supplied to steel members to ensure that their temperatures do not exceed the limiting temperatures specified in Table 2.7. There are a number of methods that are currently used to provide passive fire protection, and the three main methods are the following:

- **Thin film intumescent coatings** - Thin film intumescent coatings are reactive coating materials that undergo a chemical reaction during a fire. During this reaction, intumescent paint swells to around 50 times its original thickness and insulates the steel section beneath it. Intumescent paints are frequently used both internationally and in South Africa, and are able to follow any steel profile. They are aesthetically pleasing, but tend to become very costly as the fire rating exceeds 60 minutes. An additional consideration when using intumescent paints is whether or not on-site or off-site application will take place. Both forms of application have advantages and disadvantages.

- **Vermiculite sprays** - Vermiculite sprays, or plasters, work similarly to intumescent paint in that they insulate the steel member beneath it during a fire. They are unreactive, are applied more thickly than intumescent paints, and do not result in an aesthetically pleasing finish. Vermiculite sprays are typically the cheapest form of fire protection, and high fire ratings can be achieved economically.

- **Fire resistant boards** - Protective boards are typically gypsum-type boards which can be placed around steel sections. Unfortunately, there are currently no boards that are manufactured in South Africa, and as such these boards need to be imported. Fitting the boards is a dry trade, and it therefore allows other trades to take place in the vicinity at the same time. The same cannot be said for the on-site application intumescent paint and vermiculite sprays, which both need to take place in isolation from other building activities.

Figure 2.15 reveals the prevalence of different passive fire protection methods in Great Britain over the past 15 years.

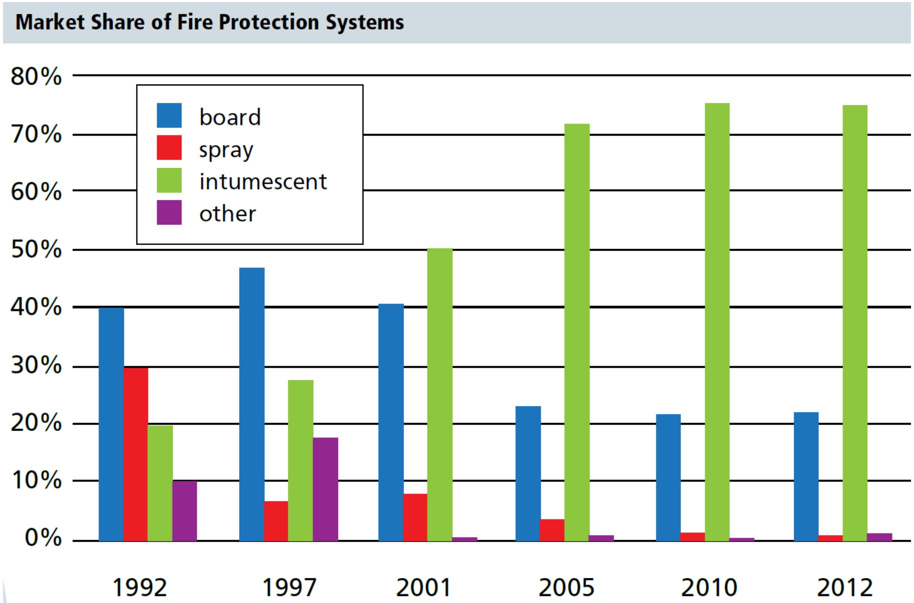


Figure 2.15: Market share and trends of fire protection in Great Britain for the past 15 years (Barrett Byrd Associates, 2013)

Figure 2.15 reveals that intumescent paint is the dominant form of fire protection in Great Britain, with a market share of 75 % in 2012. Of this 75 % market share, 55 % is applied on-site, with the remaining 25 % being applied off-site (Steel Construction Institute, 2016). Details regarding on-site and off-site application of passive fire protection are discussed below.

- **Off-site** - Off-site application involves applying fire protection in the fabrication shop, prior to the steel arriving on site. An advantage of this is that fire protection is applied in a controlled environment, which enables improved quality control. Additionally, steel arrives on site ready to be erected, which means no additional trades are required, and can assist in reducing construction time. Off-site application tends to be used where a non-aesthetic finish is required, because avoiding damage to the fire protection during delivery and erection is challenging, and repair work will often be required. Achieving the required aesthetic finish during repairs is also difficult. Off-site intumescent application is typically used where the programme savings offset the additional work required in the fabrication shop, and the majority of commercial buildings in London make use of this method (Barrett Byrd Associates, 2013).

- **On-site** - On-site application involves applying fire protection once the steel has arrived on site. Accessing the steel members may prove to be challenging due to the fact that they may already be lifted in place. Quality control is also made more difficult because application no longer occurs in factory conditions. On-site application adds an additional trade to the construction programme which could influence the ability of following trades to commence. On-site application allows for aesthetic applications of steel to be achieved, and can assist in reducing the lead-in time by eliminating the need for steel to receive fire protection prior to arriving on site.

2.7.3.3 Section factors

A critical consideration when analysing the performance of steel sections in fire, is the section factor. The section factor is a measure of the rate at which a steel section heats up during a fire. It is defined as the surface area of the member per unit length (A_m) divided by the volume per unit length (V) (Association for Specialist Fire Protection, 2014). The section factor can also be expressed as the heated perimeter of the exposed cross section (H_p) divided by the total cross section area (A). Both definitions give the same value and are measured in units m^{-1}. Section factors can range from 25 m^{-1} for very large, stocky sections to 300 m^{-1} for small, slender sections. Section factors take account of the number of exposed sides of the steel. For example, a steel beam supporting a concrete slab will only be exposed on 3 sides, whereas an internal column will typically be exposed on 4 sides. Figure 2.16 shows an example of two members with a low and high section factor respectively. The primary reason for considering the section factor is that it influences the amount of fire protection that is required, which directly influences the required cost of the fire protection.

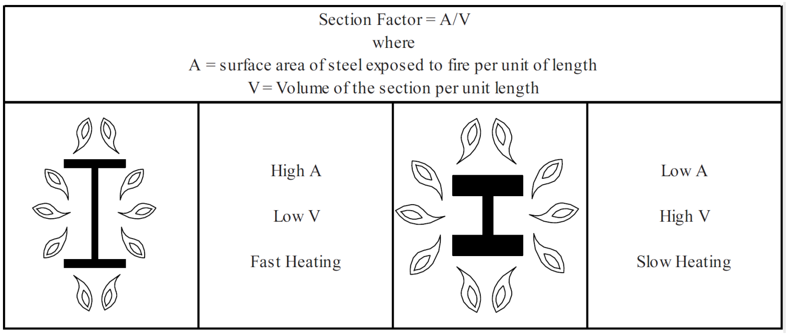


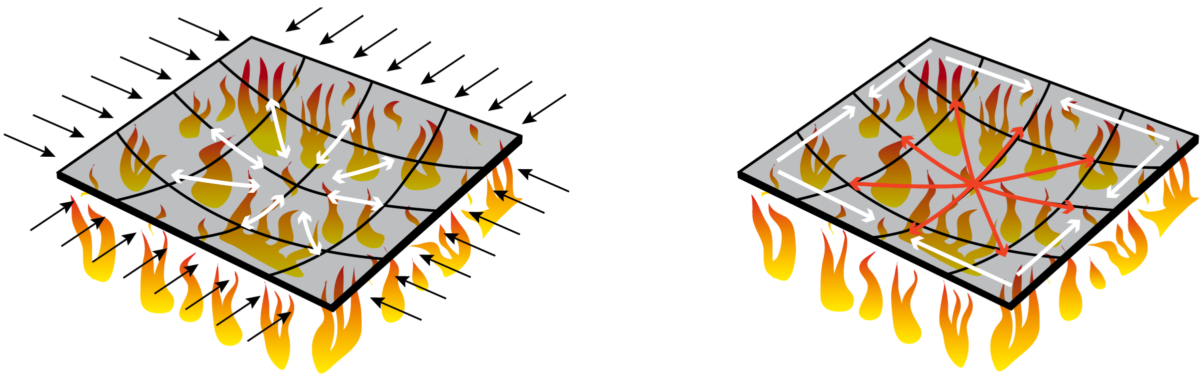
Figure 2.16: Comparison of steel members with a high and low section factor (Association for Specialist Fire Protection, 2014)

2.7.4 Behaviour of composite steel deck floors under fire

The fire design of composite steel deck floors has received a great deal of research in recent years, with the catalyst for much of this research being a fire at Broadgate Phase 8 in 1990 (Barrett Byrd Associates, 2013). The Broadgate structure consisted of composite steel deck floors with partially applied fire protection, and was subjected to a four hour fire. According to fire design knowledge available at the time, the structure should have collapsed. Not only did the structure not collapse, but it was able to come through the fire relatively unscathed. This revealed that composite steel deck structures could potentially possess a significantly greater fire resistance than was appreciated at the time.

Following the fire at the Broadgate Phase 8 facility, seven full scale fire tests were conducted at the Building Research Establishment's Cardington facility from 1994 to 2003 to evaluate the fire resistance of composite steel deck structures. During these tests steel temperatures in excess

1100 °C were recorded, and still the structure did not fail. The test temperatures were far above 700 °C, which is the failure temperature of a member when tested in isolation (Barrett Byrd Associates, 2013). The tests therefore revealed that composite steel deck floors have a significantly enhanced fire resistance, which is not revealed by testing members in isolation. The fact that many of the beams were left unprotected, enabled the slab to deflect downwards, and at high temperatures, tensile membrane action was developed. The tensile membrane action was able to provide a significantly enhanced load carrying capacity. Figure 2.17 shows the development of tensile membrane action in a composite steel deck floor.



(a) Compressive arching at low deflections

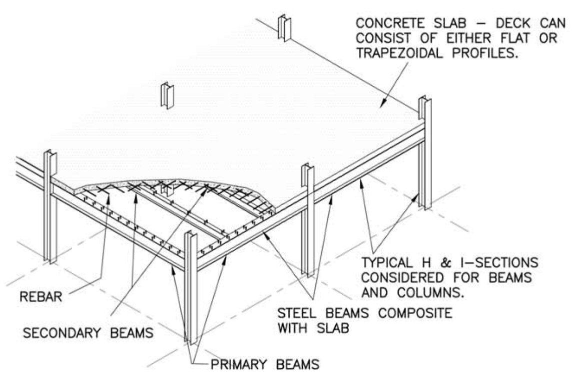
(b) Tensile membrane action

Figure 2.17: Development of tensile membrane action in a composite steel deck floor (Barrett Byrd Associates, 2013)

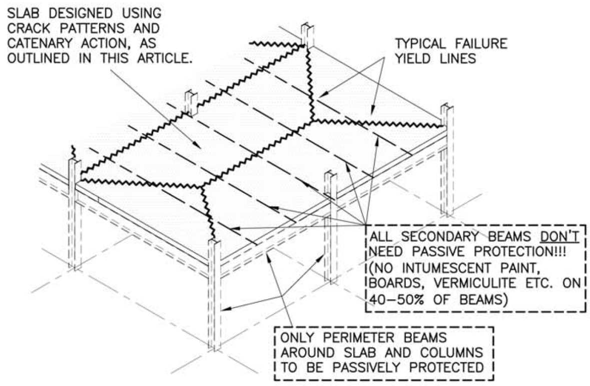
2.7.4.1 Slab Panel Method

The results of the Cardington tests sparked the research and development of a number of structural models to analyse the behaviour of composite steel deck floors during a fire. An initial method to model the tensile membrane behaviour of composite floors in severe fires, was developed by Professor Colin Bailey who validated his model with the Cardington fire tests. This method was then researched further and improved upon by Professor Charles Clifton, from Auckland University in New Zealand, who developed the Slab Panel Method (SPM) (Walls, 2015). The SPM is a structural fire design method for composite steel deck floors during severe fires (Geldenhuys and Walls, 2015).

Figure 2.18 shows an example of a composite floor layout, and the formation of yield lines during a severe fire, according to SPM assumptions. The principle behind the SPM involves only providing passive fire protection to the columns and primary beams, with secondary beams being left unprotected. This can lead to significant cost savings in the amount of passive fire protection required. The SPM utilizes the reserve strength of a floor system under deformation during a severe fire and can be compared to building design for limited to fully ductile response during earthquakes (Clifton and Feeney, 2004). There are certain detailing requirements that must be adhered to in order to use the SPM method, and these requirements are discussed in Section 3.6.4.



(a) Floor layout



(b) Yield line formation

Figure 2.18: Composite floor layout and yield line formation(Geldenhuys and Walls, 2015)

In order to use the slab panel method the following detailing requirements must be adhered to (Clifton and Feeney, 2004), (Walls, 2015):

- Floor slabs must be sufficiently ductile to allow for development of plastic hinges.
- Passive fire protection should only be applied to columns and primary beams with secondary beams remaining unprotected.
- Steel connections must allow for sufficient inelastic rotation.
- Interconnection between floor slab and steel beams must be designed as specified in technical reports.
- Reinforcement around slab panel edges must allow sufficient connection between slab and beam.

A study by Geldenhuys (2014) revealed the Slab Panel Method to be suitable for application in South Africa, with only minor changes to the fire loading being required to allow it to match the requirements in the South African code. Therefore for the purposes of this study, the SPM was employed in the design of the composite steel structure with metal deck floors. Permission was obtained from Auckland University in New Zealand to make use of SPM software, and the results of the SPM design are presented and discussed in Section 3.6.4.

2.7.5 Floor vibrations

Human induced floor loads constitute a large portion of the imposed loads for office buildings, and because they often act as dynamic loads, they can lead to excessive floor vibrations from a human comfort viewpoint (Costa-Neves et al., 2014). Furthermore, modern architectural and structural engineering trends favour buildings with large uninterrupted floor areas, that are comparatively light and flexible. These trends have served to increase the awareness of the dynamic performance of floors when subjected to human induced loading (Smith et al., 2009).

After construction of a floor system it is very difficult to reduce its susceptibility to floor vibration, with only major changes to the mass, stiffness and damping resulting in any perceptible reduction in vibration amplitudes. It is therefore important that acceptable vibration characteristics are ensured from the outset, with special attention being paid to the intended use of the floors (Smith et al., 2009). Annex C of the South African Steel Code, SANS 10162-1 (*The structural use of steel, Part 1: Limit-states design of hot-rolled steelwork* 2011), provides information regarding floor vibrations. It specifies that the specific vibration characteristics of a floor should be evaluated during the design to ensure that disturbing floor vibrations do not occur under normal human activity. Additionally, it specifies general requirements that must be considered during the design of the floor, with some of these aspects including:

- The characteristics and nature of the forcing excitations
- The acceptance criteria for human comfort
- The determination of the natural frequency of the floor framing systems, including the effect of continuity
- The modal damping ratio
- The effective floor panel weights

Although SANS 10162-1 does not provide a specific design methodology to evaluate whether floor vibrations are of an acceptable level, it does refer the reader to a number of additional resources. One of these resources is a design guide published by the American Institute of Steel Construction (AISC) entitled Floor Vibrations Due to Human Activity (Murray et al., 2003). The methodology specified in this resource was used to ensure that floor vibration criteria were satisfied for the steel framed buildings considered in this study.

2.7.5.1 Acceptance criteria for human comfort

The acceptance criteria for floor vibrations depends greatly on the intended use of the floor structure. For example, people who are taking part in activities such as dancing or aerobics, will accept vibrations 10 times greater than people in an office environment. It is therefore important to establish the intended use of a structure, so that the appropriate vibration acceptance criteria can be selected. The recommended peak acceleration values, from a human comfort viewpoint, for vibrations for various building uses is presented in Figure 2.19. For an office structure the recommended peak acceleration value is 0.5 % times the acceleration due to gravity.

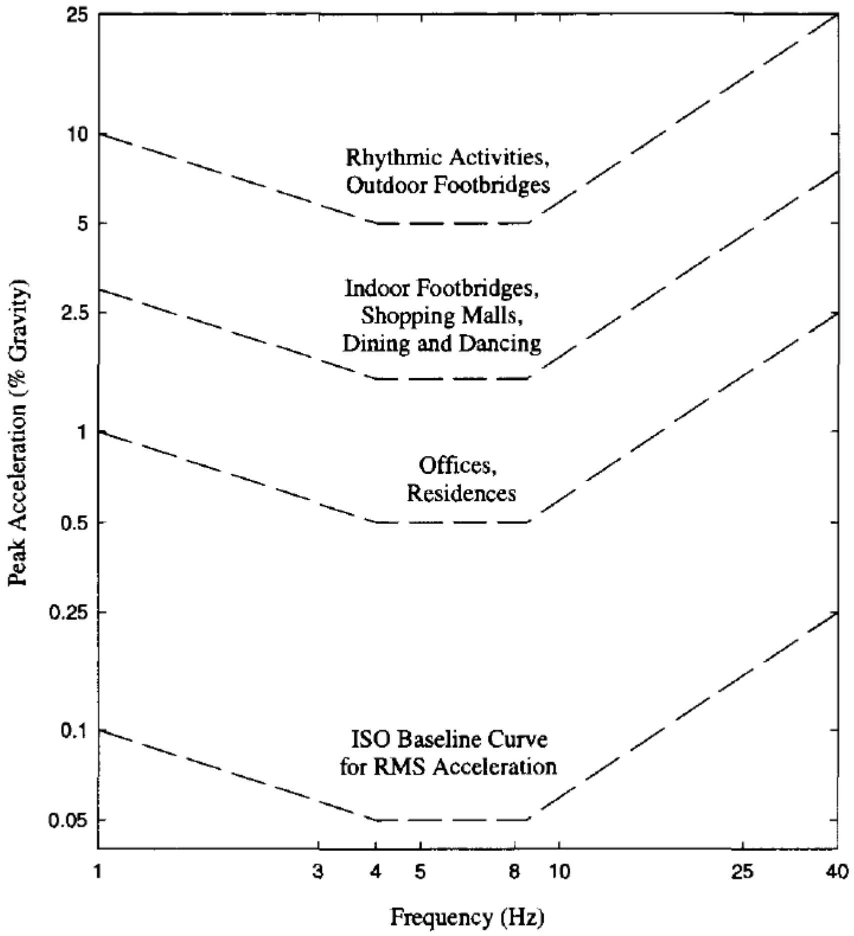


Figure 2.19: Recommended peak acceleration for human comfort for vibrations due to human activities (Murray et al., 2003)

2.7.5.2 Design for walking excitation

The most frequent and important internal source of dynamic excitation is pedestrian traffic (Smith et al., 2009). A person walking at a consistent speed applies a periodic response which can cause the floor to resonate. Murray et al. (2003) recommended a simplified design criterion for the design of floors under walking excitation this criterion is shown in Equation 2.1.

$$\frac{a_p}{g} = \frac{P_0 * exp(-0.35 * f_n)}{\beta * W} \leq \frac{a_0}{g} \tag{2.1}$$

The recommended values for various parameters and building occupancies are shown in Table 2.8.

Table 2.8: Recommended values of parameters and acceleration limits (Murray et al., 2003)

Intended building use	Constant force P_0 [kN]	Damping ratio β	Acceleration limit a_0/g x 100%
Offices, Residences, Churches	0.29	0.02 - 0.05 *	0.50%
Shopping Malls	0.29	0.02	1.50%
Indoor footbridges	0.41	0.01	1.50%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions etc.) as can occur in open work areas or churches.

0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular offices.

0.05 for full height partitions between floors.

2.7.5.3 Natural frequency of steel framed floor systems

The most important parameter when evaluating floor systems for vibration serviceability design is natural frequency (Murray et al., 2003). The natural frequency is a measure of the rate at which a system vibrates, and is an important consideration when attempting to predict the effect that external forces will have on a floor system. The lowest natural frequency, or fundamental frequency, is typically of most concern.

Steel framed floors generally have two-way systems and several vibration modes can have very similar frequencies. The procedure that is employed in this study was developed for a floor that consists of a steel beam supporting a concrete deck. The natural frequency is estimated by first considering a beam mode and a girder mode separately, and then combining them. In order to take the composite action between the concrete slab and the supporting beam into account, the transformed moment of inertia is to be used when calculating the natural frequency of the floor system. The results of the design of both the steel composite structure and steel hollowcore structure for floor vibrations are presented in Section 3.6.3 and Section 3.7.3

Chapter 3

Structural development and design

This purpose of this chapter is to describe the design methodology that was followed during the development of the steel and concrete structural alternatives that were considered in this study. The chapter begins by describing the floor layout and building configuration that was chosen to represent a typical office structure. Next, a description of different structural alternatives is included, in addition to the design loads and limit states that were considered during the design process. Critical details regarding the design of each of the structural alternatives are provided. Additional information regarding the design calculations that were performed, and the results thereof, are included in Appendix A.

3.1 Development of structural configuration

The aim of this study is to compare the cost of steel and concrete structural alternatives when used for the structures of a typical low-rise multi-storey office building. Therefore, the building configuration was developed in such a way that it could be seen as being representative of a typical low-rise office building in South Africa. This was achieved through research into typical requirements for office structures, in addition to investigating various office building layouts currently used in South Africa. Furthermore, meetings were held with a consulting engineer with many years of experience in the design of multi-storey commercial structures, to obtain insight into layouts that are frequently employed in such structures. This enabled a layout to be developed that could be regarded as "typical".

Through the research and meetings, two building configurations were developed for the purpose of this study. The floor layouts of the buildings are similar, the only difference being the removal of the internal columns in one instance to result in a longer spanning structure. For the purposes of this study the resulting buildings will be referred to as the short span and long span structure respectively. The building layouts are shown in Figure 3.1 and Figure 3.2.

3.1.1 Short span layout

The floor layout and elevation of the short span structure is shown in Figure 3.1. The width of the building is 13 m and the length 37.5 m, which consists of five equal 7.5 m spans. This results in a gross area of 487.5 m^2 per floor, and a total floor area of 1950 m^2. The floor-to-floor height varies for each of the structural solutions, but is approximately 3.5 m, which results in a total building height of 15 m on the high end of the structure considering the slope of the 5° mono-pitch roof.

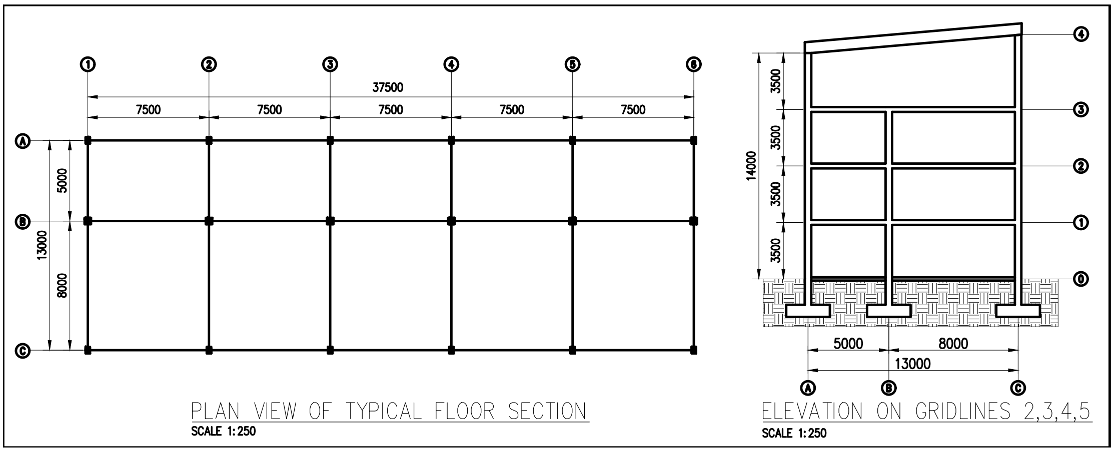


Figure 3.1: Short span building configuration

3.1.2 Long span layout

The long span floor layout used in the study is shown in Figure 3.2. It is very similar to the short span layout, the only difference being the absence of the internal columns along Gridline B. This results in the floor system needing to span the full 13 m width of the building. The advantage of this layout is the provision of uninterrupted, column free floor space, which is attractive from a client's point of view as it offers greater flexibility in the building's use. The floor-to-floor height of the building can remain relatively similar to the short span structure due to the integration of services within the depth of the floor system.

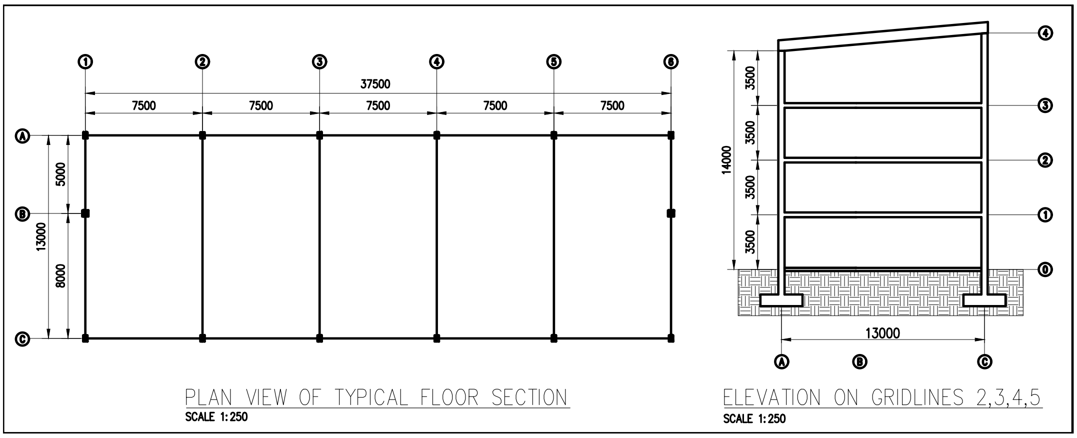


Figure 3.2: Long span building configuration

3.1.3 Development of floor layout

Several factors were considered during the development of the floor layout for the building used in this study. Some of the main considerations were:

- **Mixed occupancy floor space** - Mixed occupancy floor space is frequently required in multi-storey office buildings. This often involves parking space at lower levels with office space at higher levels. Many multi-storey office buildings therefore require a column grid that is compatible with different building uses (Tata Steel, SCI and BCSA, 2015b). The column grid and floor layout used in this study were chosen in such a way that they allow for basement parking if required, with office space above. The regular 7.5 m grid enables three cars to be parked alongside one another, with the 5 m span being suitable for the length of a parking bay, while the 8 m span allows sufficient space for the manoeuvring of cars.

- **Building width** - Naturally lit and ventilated zones extend a distance of twice the floor-to-ceiling height from the outer walls (Steel Alliance, 2010). Therefore, the 13 m width of the building allows for good natural light and ventilation (Hicks et al., 2004). Furthermore, steel sections are available in standard lengths of 13 m, and as such, a 13 m span is efficient when ordering steel for the long span structure.

- **Location and site constraints** - The building is regarded as being located in an out-of-town or suburban location and is not dictated by site constraints, such as neighbouring buildings or ground conditions. A regular, rectangular building footprint was therefore chosen which allows for an efficient use of floor space. Furthermore it results in a high degree of repetition and standardisation which simplifies the design and construction of both the steel and concrete structural alternatives.

- **Building height** - Office buildings in suburban locations typically have fewer stories than those located in city centres. This is because space is often limited in city centres, with buildings being constrained by those around them (Tata Steel, SCI and BCSA, 2015b). In addition, land costs tend to be higher in city centres, so providing space using taller structures is often the preferred solution. In contrast, office buildings situated in out-of-town locations are often not dictated by neighbouring structures, and land costs also tend to be cheaper. For these reasons, low to medium rise office structures tend to be preferred in out-of-town locations. Four stories were therefore identified as an efficient number of stories for the building considered in this study

3.2 Steel and concrete structural alternatives

Figure 3.3 illustrates the structural alternatives that were considered in this study.

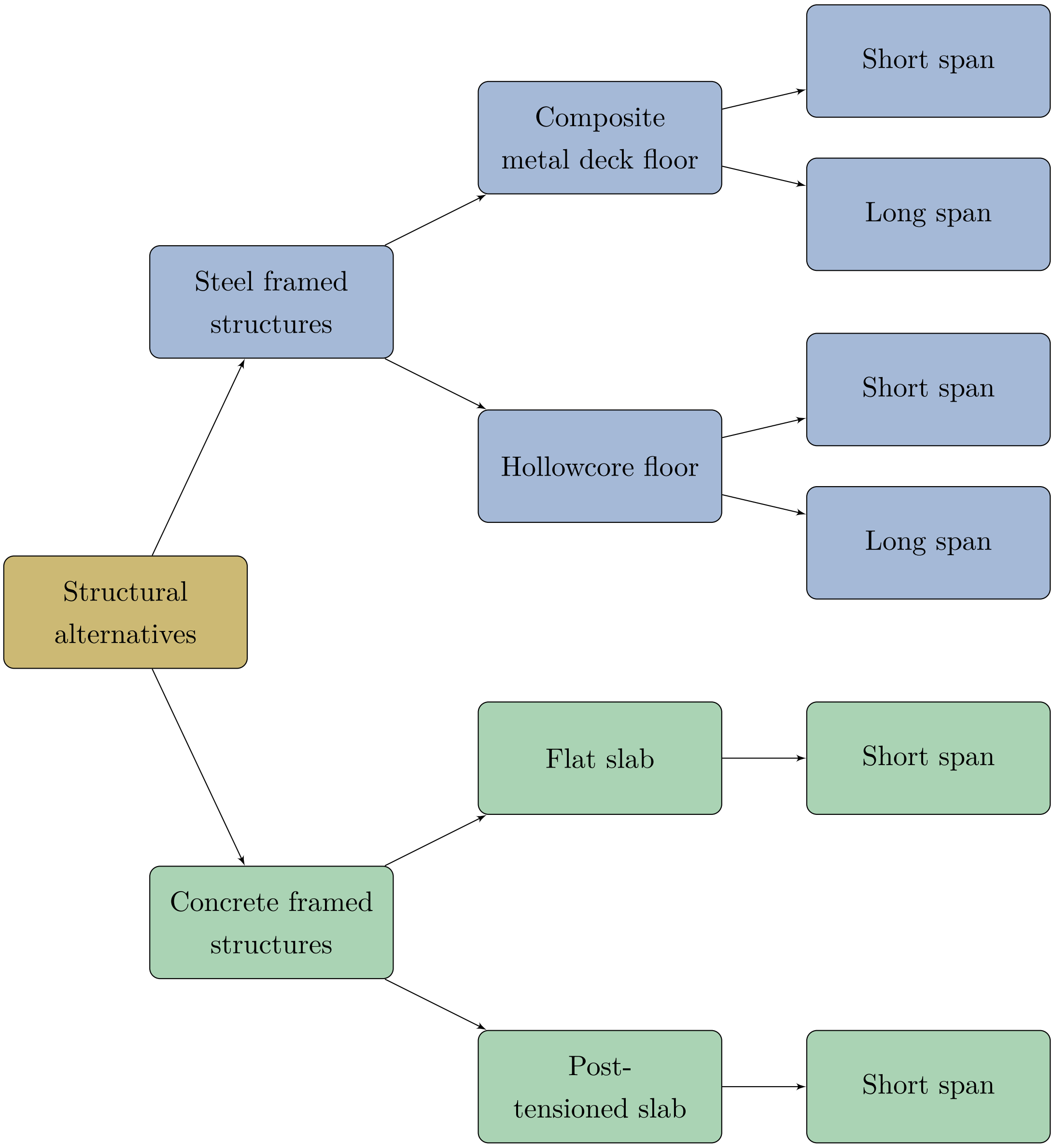


Figure 3.3: Steel and concrete structural alternatives considered in this study

The steel and concrete structural alternatives were chosen to reflect current building practice in South Africa, and represent options that are available when selecting a structural solution for a project. For both the steel and concrete framed buildings, two different floor systems were considered. This was extended further in the case of the steel framed structures, where both a short and long span configuration were considered for each of the floor systems, as shown in Figure 3.1 and 3.2. A short description of each of the floor systems is provided below, with a more detailed description, and design calculations, included in Sections 3.6 to 3.9 later in this chapter.

- **Composite metal deck building-** The composite metal deck floor system consists of steel beams supporting a profiled steel sheeting and reinforced concrete slab. The slab is structurally tied to the supporting steel beam by welded shear studs. 0.8 mm thick Bond-Dek profiled floor sheeting acts as permanent formwork. Both propped and unpropped construction were considered in this study (See Section 2.4.1.1).

- **Steel beams supporting hollowcore slabs** - This floor system consists of steel beams supporting hollowcore floor slabs, which are one-way spanning precast and prestressed concrete slabs with longitudinal hollowcore cores (See Section 2.4.1.2). Both composite and non-composite design of the steel beams and slabs have been considered.

The following two floor systems are considered for the concrete frame structures:

- **Reinforced concrete flat slab with edge beams** - A flat slab floor system transfers loads directly to columns, without any beams being required (See Section 2.4.2. They are a popular construction method for commercial structures in South Africa and their design and construction is well understood among industry professionals. Edge beams are required to support the cladding load and assist in overcoming problems associated with punching shear at perimeter columns (Goodchild, 1997).

- **Post-tensioned flat slab with edge beams** - The second concrete option is a reinforced concrete structure with post-tensioned floors (See Section 2.4.3. Unbonded tendons were used in the post-tensioned floor, and edge beams were required to support the cladding load. Post-tensioned concrete floor systems are a popular construction method currently in use in South Africa.

3.3 Design loads and limit states

3.3.1 Loading

The following loads were considered during the design of each of the structural solutions:

- **Permanent load** - The permanent load consists of the self weight of each structural system, as well as an additional 0.5 kPa to allow for the weight of the ceiling and services. The magnitude of the load to account for the ceiling and services was discussed and agreed upon with a consulting engineer with many years of experience in the design of commercial structures. The value of 0.5 kPa was deemed to be sufficient for a wide range of commercial buildings and was be used for all building options. This was confirmed by Brown et al. (2009) who recommends a typical self weight of 0.25 kPa for services and 0.1 kPa for ceilings which results in a total weight of 0.35 kPa. An additional load could be required for a raised floor, which would result in a total load very close to the 0.5 kPa that was chosen.

- **Imposed load** - The imposed load is made up of the imposed floor load and the weight of movable partitions. The magnitudes of the imposed loads have been obtained from SANS 10160 - Part 2: Self-weight and imposed loads (*Basis of structural design and actions for buildings and industrial actions* 2011a), which recommends an imposed floor load of 2.5 kPa for offices for general use. An additional 1 kPa was used to take into account the weight of movable partitions, resulting in a total imposed load of 3.5 kPa.

- **Wind load** - The wind load on the structure was calculated according to SANS 10160 - Part 3: Wind actions (*Basis of structural design and actions for buildings and industrial actions* 2011b). The wind load has been considered for wind blowing in two directions. Sufficient lateral stability is provided for both the steel and concrete structures, for both critical wind directions.

3.3.2 Limit states

The following limit states were considered while designing the buildings. The corresponding load factors and combinations were obtained from SABS Part 1: Basis for structural design (*Basis of structural design and actions for buildings and industrial actions* 2011c).

- **Ultimate limit state (ULS)** - SANS 10160-1 describes the ULS as the "state associated with collapse or with similar forms of structural failure". The ULS has been considered for the required load combinations of permanent, imposed and wind loads, in order to ensure structural failure does not occur.

- **Serviceability limit state (SLS)** - SANS 10160-1 describes the SLS as the "states that correspond to condition beyond which specified service requirements for a structure or structural member are no longer met". In order to ensure that SLS criteria were satisfied two aspects were considered:

  - *Deflections* - A deflection limit for vertical deflection of $\Delta < \frac{1}{300}$ was used, which is the recommended limit from Annex D of SANS 10162-1 (*The structural use of steel, Part 1: Limit-states design of hot-rolled steelwork* 2011).

  - *Floor vibrations* - Floor vibrations under human induced loading have been considered in accordance with the requirements of SANS 10162-1, Annex C. It was ensured that the steel framed structures met the criteria specified in this Annex.

- **Fire limit state (FLS)** - Table 5 of SANS 10400, Part T (*The application of the National Building Regulations* 2008) specifies the required fire rating for a range of building types and uses. From this table it was determined that for the building considered in this study, a 60 minute fire rating is required. The meaning of this rating has already been discussed in Section 2.7.2. All passive fire protection in this study has been applied to ensure that a 60 minute fire rating is achieved.

3.4 Components of structure to develop for comparison

A number of components were considered while developing the comparison between the various structural alternatives. The following section explains some of the decisions that were made with regard to the design of various structural components.

3.4.1 Building foundations and substructure

The foundations for each of the structural solutions were designed using reinforced concrete pad footings. The footing dimensions were calculated in detail for each of the structural alternatives. The difference in self-weight between the various structural alternatives resulted in different footing sizes being required. The comparison of the footing sizes for each of the steel and concrete structural alternatives is discussed in greater detail in Section 3.10 later in this chapter.

Due to the fact that the concrete framed structures make use of a brick and mortar cladding system, a strip footing is required along the perimeter of the structure to support the cladding load at ground level. The steel framed structures are assumed to be constructed with a light steel framing cladding system, and due to the reduced weight of this system, no strip footing is required. However, the influence of the structural envelope was not included for the purposes of this comparison. As such, the cost and programme implications of the strip footing have not been considered in this study.

The ground floor slab is the final component of the building foundations and substructure that was considered. The construction time and cost of the ground floor slab were assumed to be the same for all building options.

3.4.2 Building frame and floors

The steel and concrete frame as well as the various floor systems were designed in detail for each of the structural alternatives. This involves all aspects of the building's frame including columns, beams, slabs, shear connection, and any other components making up the frame of the building.

3.4.3 Building envelope

The following cladding systems were employed in the different structural options:

- **Steel framed buildings** - The cladding used in the steel framed buildings was a light steel framing (LSF) cladding system. There is currently a growing trend among leading South African architects to make use of LSF for the curtain walls of multi-storey office buildings (Barnard, 2015). A light steel framing weight of 35 kg/m^2 was used for the design of the supporting structure.
- **Concrete framed buildings** - The cladding system employed in the concrete buildings consisted of masonry units and mortar. This method is very commonly used in South Africa for concrete framed office buildings.

For the purposes of this study it was assumed that the construction time and cost of the different cladding systems were the same. In reality, there would be cost and programme differences associated with using each of the cladding systems, but these have not been considered in this study.

3.4.4 Non-structural components

The study focuses on the comparison between the building frames for the various steel and concrete structural alternatives. It was therefore assumed that the construction time and cost of the non-structural components were constant across all of the structural alternatives. The non-structural components that were considered in this study include:

- **Roof** - The form of the roof was taken as a mono-pitch steel roof truss spanning the full 13 m width of the floor.
- **Mechanical and electrical services** - Mechanical and electrical services consist of air-conditioning and the provision of typical electrical requirements for an office structure.
- **Finishes and fitments** - The finishes and fitments were assumed to be those encountered in a typical office building in South Africa. This includes aspects such as carpets, partitions and doors, to name a few.
- **Parking space around the building** - For a typical office building 4 parking bays are required per 100 m^2 of floor area. No basement parking was provided so sufficient parking needs to be provided around the office building.
- **Plumbing and wetpoints** - 25 m^2 of plumbing and wetpoint area have been assumed to be provided per floor.
- **One lift** - One lift is provided in all structural options.

Although the focus of this study is the comparison of the building's frame, the total construction cost of the building is still an important consideration, as will become clear in Chapter 5 where the cost analyses and comparisons are discussed.

3.5 Steel framed structures

This section provides information relating to the design of the steel framed structural alternatives. Particular attention was given to key considerations during the design process that would influence the cost of the structure. These issues are discussed in detail for both the steel composite and hollowcore options.

3.5.1 General / Typical design decisions

Some of the design decisions relating to the steel building options include the following:

- **Steel sections** - All steel sections listed in the South African Steel Construction Handbook (South African Institute of Steel Construction, 2013) were considered to be available for use in the structure. H-sections were used for columns, and I-sections and plate girders for beams.

- **Connections** - For both steel framed structural alternatives, all connections were assumed to be nominally pinned, or simple connections. Such connections transfer a shear force to the supporting member, and only a small moment is induced due to the eccentricity of the connection (De Clercq et al., 2012). The use of simple connections simplifies the design and fabrication process of the steel members. Furthermore, simple connections allow light column sections to be used because of the relatively small moment that is transferred to the column. Bolted connections were also used wherever possible in the structure. This enables rapid erection of the frame once the steelwork arrives on site.

- **Column splices** - Steel column sections typically span over 2 stories before being spliced. The Green Book (De Clercq et al., 2012) advocates the use of standard bolted column splices. Therefore all splice connections in the steel framed buildings were assumed to be provided using standard splice details from the Green Book. Columns were spliced at a position 1 m above floor level to enable easy access during construction.

- **Lateral restraint** - Lateral restraint was provided in the form of braced steel bays which brace the structure in both directions. Angle sections bolted to gusset plates were used for the bracing.

- **Fire protection** - The provision of fire protection is a particularly important consideration in the design of multi-storey steel structures. The structures considered in this study require a one hour fire rating as specified in Table 5, Part-T of SANS 10400 (*The application of the National Building Regulations* 2008). Passive fire protection was provided through the on-site application of vermiculite spray. The costs of both intumescent paint and vermiculite spray are discussed in Section 5.1.2.2. The application of the vermiculite spray would follow directly after the erection of the steel frame and floors.

3.6 Design of steel framed building with composite steel deck floor

3.6.1 Layout and loading details

The first steel framed structural alternative that is discussed is the steel framed structure supporting composite metal deck floors. The short span floor layout as well as the required composite beam sizes for unpropped construction, are indicated in Figure 3.4.

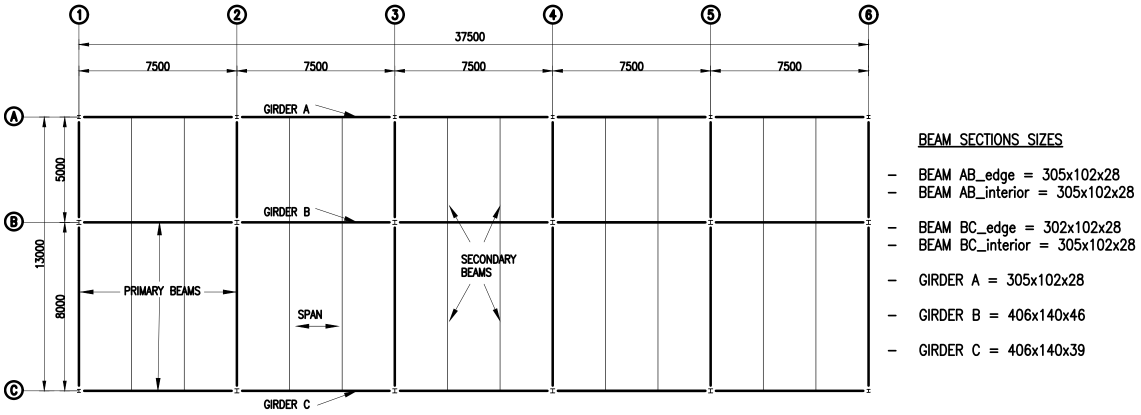


Figure 3.4: Floor layout of short span steel composite building

0.8 mm thick Bond-Dek steel sheeting was used to provide permanent formwork for the concrete slab. The sheeting was 75 mm deep and supported a cover slab of 65 mm, which resulted in a total slab depth of 140 mm. Due to the limited spanning capability of the Bond-Dek sheeting, secondary beams are spaced at 2.5 m. All beams and girders have been designed to act compositely with the slab through the use of welded shear connectors. 125 mm long, 19 mm diameter shear studs were used with 2 studs welded onto the beam's flange at each trough position in the metal deck, which implies a spacing of 450 mm. The total number of shear studs required for the short span composite building was 4380. 25 MPa concrete was used for the concrete slab, and the top surface of the concrete received a power float finish. An additional 15 % of the concrete weight was added to the permanent load to take into account ponding of concrete.

The long span floor layout was very similar to the short span layout, the only difference being that the composite beams were now required to span the full 13 m floor width. There was no change to the Bond-Dek sheet and slab details, and consequently secondary beams were still required at a spacing of 2.5 m. As for the short span structure, 2 shear studs were provided in each trough of the Bond-Dek sheet. The total number of shear studs required for the long span composite structure was 3900. The long span floor layout, as well as the required beam sizes for unpropped construction are shown in Figure 3.5.

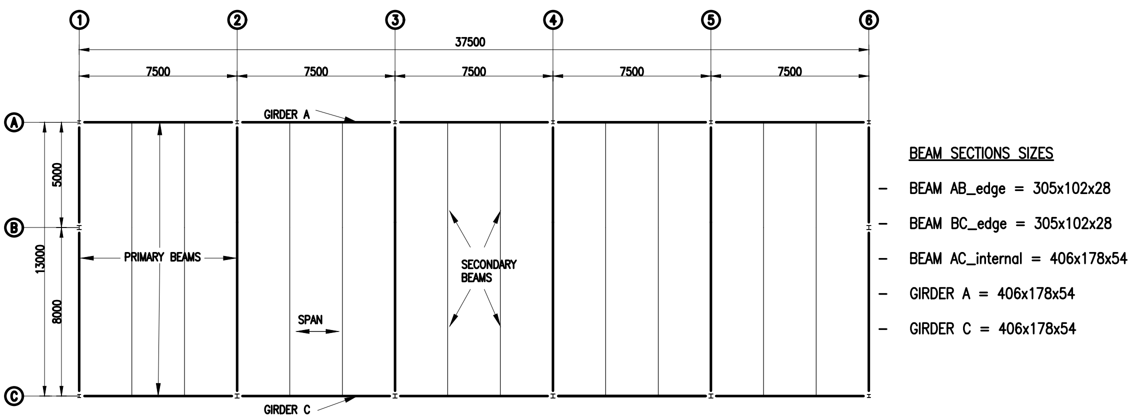


Figure 3.5: Floor layout of long span steel composite building

3.6.2 Design of internal composite beam

The purpose of this section is to discuss the design of a typical internal composite beam and to illustrate some of the factors that were considered when selecting a beam section. A typical 8 m internal beam is shown in Figure 3.6 and the design of this beam is discussed in this section. Furthermore, an example of the calculation procedure that was followed during the design of this beam has been included in Appendix A.

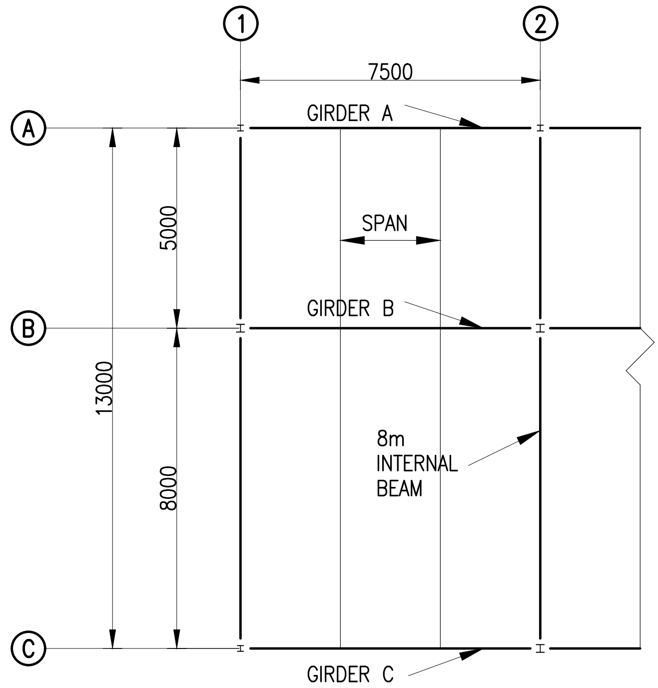


Figure 3.6: Section of floor layout indicating beam used for design discussion

The 8 m beam considered discussed in this section was simply supported at both ends, and designed on the basis of a partial shear connection of 60 %. As mentioned previously in Section 2.4.1.1, a fundamental decision that has to be made when designing composite beams supporting metal deck, is whether propped or unpropped construction will be used. This decision will most likely influence the size of the steel beams that are be used, due to the different design checks that are required. The moment resistance, as well as the deflection of various I-sections when designed as composite beams, are presented in Table 3.1 and Table 3.2, to provide insight into how the beam section was selected.

Table 3.1 illustrates that for the 8 m internal composite beam, almost all I-sections were able to satisfy the moment resistance requirements. Sufficient moment resistance was provided even through the use of a partial shear connection. This is because a large reduction in the percentage of shear connection supplied, can only result in a small reduction in flexural resistance. Typically, composite beams with a partial shear connection of 40 % are still able to attain 85 % of the flexural capacity that is achieved when a full shear connection is provided (South African Institute of Steel Construction, 2013).

Table 3.1: Moment resistance of various I-sections when used for a typical 8 m internal composite beam

SECTION DESIGNATION	CONSTRUCTION STAGE						COMPOSITE STAGE		
	Placing of deck			Placing of concrete			Composite beam		
	M_u	M_r	Check	M_u	M_r	Check	M_u	M_r	Check
h x b x kg/m	kNm	kNm		kNm	kNm		kNm	kNm	
203x133x25	20.49	51.00	OK	106.04	82.75	NOT OK	198.04	200.08	OK
203x133x30	20.49	68.20	OK	106.04	100.00	NOT OK	198.04	220.38	OK
254x146x31	20.49	81.80	OK	106.04	126.20	OK	198.04	254.95	OK
254x146x37	20.49	109.00	OK	106.04	154.96	OK	198.04	286.96	OK
254x146x43	20.49	135.00	OK	106.04	181.48	OK	198.04	319.05	OK
305x102x25	20.49	28.80	OK	106.04	107.35	OK	198.04	247.48	OK
305x102x28	20.49	40.50	OK	106.04	130.36	OK	198.04	274.46	OK
305x102x33	20.49	53.80	OK	106.04	153.68	OK	198.04	302.45	OK
305x165x40	20.49	147.00	OK	106.04	200.01	OK	198.04	342.85	OK
305x165x46	20.49	175.00	OK	106.04	230.68	OK	198.04	378.80	OK
305x165x54	20.49	212.00	OK	106.04	269.34	OK	198.04	425.62	OK
356x171x45	20.49	176.00	OK	106.04	246.97	OK	198.04	411.58	OK
356x171x51	20.49	212.00	OK	106.04	285.95	OK	198.04	455.50	OK
356x171x57	20.49	246.00	OK	106.04	322.70	OK	198.04	499.09	OK
356x171x67	20.49	307.00	OK	106.04	386.60	OK	198.04	534.72	OK
406x140x39	20.49	106.00	OK	106.04	229.40	OK	198.04	404.81	OK
406x140x46	20.49	148.00	OK	106.04	284.04	OK	198.04	468.53	OK

Table 3.2 shows the deflection of various I-beams when designed as composite beams, in both propped and unpropped construction. As mentioned previously the deflection limit of the beams has been taken as the length divided by 300 ($\Delta < \frac{L}{300}$). From Table 3.2 it is evident that the deflection of many of the beams is excessive, particularly when unpropped construction is used. If propped construction is used, the lightest beam section that was able to satisfy the deflection criteria was a 305 x 102 x 33 I-section. This is significantly lighter than the lightest unpropped beam section that was able to satisfy the deflection criteria, which was a 356 x 171 x 57 I-section. However, the deflection of a composite steel beam, can be reduced using one of the three methods discussed below:

1. Using propped construction
2. Selecting a larger steel section
3. Pre-cambering

Table 3.2: Deflection of various I-sections when used for a typical 8 m internal composite beam

SECTION DESIGNATION	DEFLECTION				
	Δ_{allow}	PROPPED		UNPROPPED	
	L/300	Δ_{total}	Check	Δ_{total}	Check
h x b x kg/m	mm	mm		mm	
203x133x25	26.67	49.58	NOT OK	125.33	NOT OK
203x133x30	26.67	43.28	NOT OK	104.07	NOT OK
254x146x31	26.67	33.73	NOT OK	71.53	NOT OK
254x146x37	26.67	29.35	NOT OK	58.92	NOT OK
254x146x43	26.67	26.34	OK	51.12	NOT OK
305x102x25	26.67	32.94	NOT OK	71.64	NOT OK
305x102x28	26.67	28.83	NOT OK	59.17	NOT OK
305x102x33	26.67	25.77	OK	50.78	NOT OK
305x165x40	26.67	22.58	OK	40.77	NOT OK
305x165x46	26.67	20.56	OK	36.02	NOT OK
305x165x54	26.67	18.60	OK	31.52	NOT OK
356x171x45	26.67	17.88	OK	30.31	NOT OK
356x171x51	26.67	16.31	OK	26.70	NOT OK
356x171x57	26.67	15.15	OK	24.19	OK
356x171x67	26.67	13.58	OK	20.88	OK
406x140x39	26.67	17.08	OK	27.62	NOT OK
406x140x46	26.67	14.92	OK	24.28	OK

As mentioned in Section 2.4.1.1, and shown in Table 3.2, propped construction can significantly reduce the deflection of composite beams, however speed of construction is sacrificed. Furthermore, propped construction inhibits early access for following trades, which is a major advantage of metal deck floor systems. It would be inefficient to pre-camber beams in addition to using propped construction, as both of these activities have time and cost implications. Therefore it would make more sense to use one or the other.

The deflection of the beam could of course be satisfied explicitly in unpropped construction through the use of a larger steel section, but this results in significantly heavier steel sections. Due to the repetitive nature of the floor system and the large number of beams that are required, this would greatly increase the mass and cost of the steelwork.

The final option to reduce the deflection, is to pre-camber the beams. This increases both fabrication time and cost, but it means that speed of construction will not be inhibited. In addition, Table 3.2 shows that the benefit of pre-cambering the 8 m beam is greater than what can be achieved through the use of propped construction. The lightest beam section that satisfies the deflection criteria in propped construction is a 305 x 102 x 33, while if the beam is precambered, a 305 x 102 x 28 can be used. The repetitive nature of the floor system means that the same pre-camber will be required for many of the beams. The additional cost associated with the precambering of the beams has been considered when calculating the cost of the structural steelwork in Chapter 5.

Considering the three methods to reduce the deflection that have been discussed, it was decided to use unpropped construction and to pre-camber beams where the deflection was excessive. This enabled a rapid speed of construction to still be achieved and following trades could be integrated into the construction programme at an early stage. Furthermore, the lightest steel sections that satisfied the moment resistance criteria could be used, so long as they were pre-cambered when necessary to limit their deflection.

3.6.3 Floor vibrations

An important serviceability consideration for metal deck floors is the vibration of the floor under human induced loads. Background on floor vibrations was provided in Section 2.7.5, as well as a description of the design methodology that was followed to ensure that floor vibrations were of an acceptable level. The methodology laid out in Murray et al. (2003) was followed, with the recommended peak acceleration values for office buildings being used in the calculations. The level of vibrations were checked for both the short and long span composite structures. The results of these calculations are presented in this section. Additional information regarding the vibration calculations that were performed has been included in Appendix A.

3.6.3.1 Floor vibrations in short span structure

The analysis of the short span composite floor system showed that the peak accelerations that could be expected were on the limit of what is deemed to be satisfactory for office buildings. Based on the calculations the peak accelerations that can be expected were found to be 0.49 % of the acceleration due to gravity. This is only slightly below the 0.5 % limit specified for floors in office buildings.

3.6.3.2 Floor vibrations in long span structure

The analysis of the long span composite floor system revealed the floor vibrations to be satisfactory. The peak acceleration to be expected was calculated to be 0.35 % of the acceleration due to gravity which is well within the 0.5 % limit.

3.6.3.3 Discussion of floor vibrations analysis

The results showed that both the short and long span composite steel deck floors were able to satisfy the vibration criteria without any changes to normal design procedures being required. However, the vibrations that can be expected in the short span building were very close to the limit, which reveals the importance of considering floor vibrations. For buildings with stricter vibration criteria, such as hospital buildings for example, changes to normal design procedures may well be required to ensure that floor vibration criteria would be satisfied.

3.6.4 Fire design using the Slab Panel Method

An introduction to the Slap Panel Method (SPM) was provided in Section 2.7.4.1. Research by Geldenhuys (2014) revealed the SPM to be suitable for use in South Africa, with only minor changes being required to match the South African loading code. Therefore, the SPM was used to design the steel composite floors at the fire limit state. Permission was obtained from the University of Auckland to make use of SPM software to perform the analysis of the composite floor systems. The methodology that was employed when analysing the steel composite floors with the SPM software is described below. A summary of the results obtained from the analysis are presented in this section.

1. **Design floor load during fire** - Load factors in the fire limit state are currently outside the scope of the South African loading code, SANS10160 (*Basis of structural design and actions for buildings and industrial actions* 2011c). The applied loading at the fire limit state was therefore calculated using the load factors recommended from the Canadian fire design annex. The specified load factors are 1.0 for the dead load and 0.5 for the imposed load. The design load on the floor during the fire condition was calculated as:

$$\begin{aligned} w_{applied} &= 1.0 * G_k + 0.5 * Q_k \\ &= 1.0 * 3.8 + 0.5 * 3.5 \\ &= 5.55\ kN/m^2 \end{aligned} \tag{3.1}$$

2. **Fire load** - The fire load can either be specified in terms of a time equivalent compared to a standard fire, or it can be calculated based on the building characteristics. These characteristics include the available fire load in the compartment, which depends on the building's intended use, openings and ventilation conditions and the nature of the boundary walls and floors. The fire load density for an office building was used and a fire cell over one whole floor was considered.

3. **Selection of slab panel** - The next step in the design process was to specify the slab panels that were used in the study. Figure 3.7 below shows how the floor was divided into the various slab panels sections. The floor was split into 5 slab panels, each 13 m long and 7.5 m wide. Passive fire protection, in the form vermiculite spray, was only applied to

members along the perimeter of each slab panel. The same slab panel layout can be used for both the short and long span structure, the only difference being the size of the beams in the floor system.

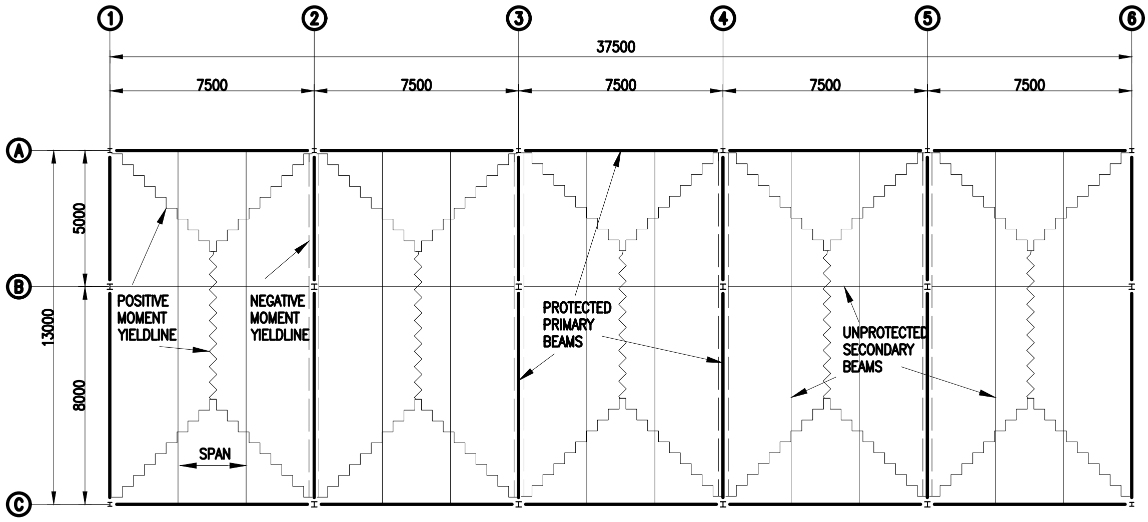


Figure 3.7: Division of floor into slab panels

4. **Reinforcement content** - The final aspect that was considered during the analysis was the amount of reinforcement that would be placed in the slab. Sufficient reinforcement must be placed in the slab to ensure that it has adequate strength and ductility during a severe fire.

Following the completion of the design steps listed above, a slab panel was modelled using the SPM software. An example of an output of the analysis with the SPM software has been included in Appendix A. The main results of the calculations can be summarised as follows

- *Design floor load* - 5.55 kPa
- *Floor capacity* - 6.71 kPa > 5.55 kPa, therefore the panel moment / tension membrane capacity is sufficient.
- *Design shear load* - 20.7 kN/m
- *Floor shear capacity* - 45.94 kN/m > 20.7 kN/m, therefore the shear capacity is sufficient.
- *Reinforcement contents* - 8 mm bars @ 200 mm spacing in both directions (Mesh ref. 395) were required. In addition to this reinforcing mesh, Y8 reinforcing bars were required in each trough of the Bond-Dek sheet, which equates to a spacing of 450 mm.

The results revealed that the floor had sufficient strength under fire conditions to form a slab panel and withstand the fire load. The only change required to normal design procedures was with regards to the detailing of the reinforcement in the slab. The reinforcement that was required was a mesh ref. 395, which consists of 8 mm bars spaced at 200 mm in each direction, in addition to a Y8 reinforcing bar in each trough of the Bond-Dek sheet. The additional reinforcement to be installed should require minimal additional construction time and effort to place.

The use of the SPM with the selection of the slab panels as shown in Figure 3.7, meant that a significant number of beams could remain unprotected. There are 47 beams per floor in the short span composite building, of which only 22 beams required fire protection when the SPM was used. This meant that 53 % of the beams required no fire protection at all, compared to having to provide passive fire protection to all the beams if the SPM was not used. There are 28 beams per floor in the long span steel composite building, and only 18 of these beams required fire protection when the SPM was used. The cost of providing fire protection in steel framed buildings constitute a significant component of the total frame cost. Therefore being able to leave a large number of the beams unprotected presents significant cost benefits. The cost of providing the fire protection and the cost benefits of using the SPM are presented in Chapter 5.

3.6.5 Calculating thickness requirements of passive fire protection for steel beams

Through the use of the SPM it was determined which steel beams required passive fire protection. The procedure that was followed when calculating the thickness of the passive fire protection is shown in Figure 3.8.

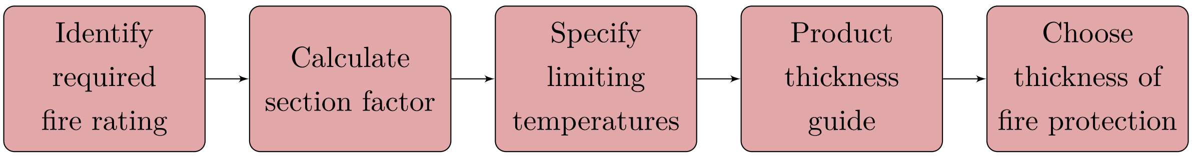


Figure 3.8: Methodology for calculating thickness of passive fire protection that is required

In order to calculate the required fire rating, Table 5 of Part-T in SANS 10400 was used, and for the office structure in this study a 60 minute fire rating was required. Next, the section factors were calculated based on the dimensions of the steel section, and the number of sides it receives heat from during a fire. The limiting temperatures that were used for beams and columns are presented in Section 2.7.3. Once the limiting temperature has been established, a product guide for the fire protection material was used to determine the thickness of the fire protection that must be supplied. The thickness depends on the duration of the fire rating, section factor and limiting temperature.

3.7 Design of hollowcore floor system supported on steel beams

3.7.1 Layout and building details

The second steel structural alternative to be discussed is the steel framed structure supporting precast hollowcore units. The short span floor layout, as well as the required beam sizes are shown in Figure 3.9.

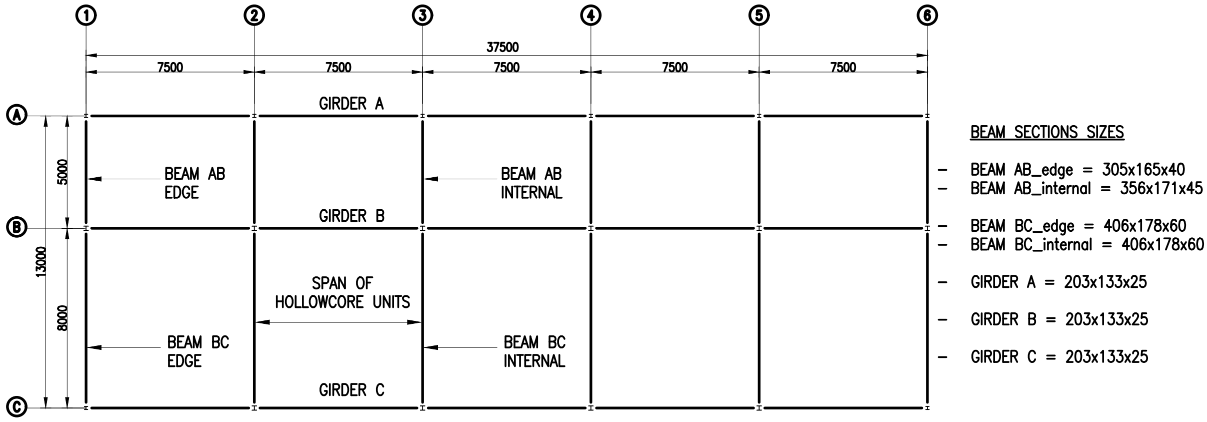


Figure 3.9: Short span floor layout in steel building supporting hollowcore units

As can be seen from Figure 3.9 the hollowcore units were able to span the 7.5 m between supporting beams. Therefore, no secondary steel beams were required for this configuration. Hollowcore units are one-way spanning precast concrete slabs and, as such, only the beams along gridlines 1 to 6 support the floor load. Girders along gridlines A to C were simply required to provide structural rigidity to the frame and support the cladding load from the light steel framing cladding. The design of hollowcore slabs was carried out by a specialist company and it was specified that in order to span the 7.5 m between the supporting steel beams, a 200 mm deep hollowcore unit was required. Following erection, a 40 to 50 mm thick levelling screed was laid, which consists of a 1:4 mix by volume of cement to clean river sand. A light reinforcing mesh was also included to control cracking of the screed.

The long span floor layout is very similar to the short span layout, the only difference being that the steel beams are now required to span the full 13 m floor width. There is no change to the hollowcore slab details as the span between supporting steel beams is still 7.5 m. The long span floor layout as well as the required beam sizes are shown in Figure 3.10 below.

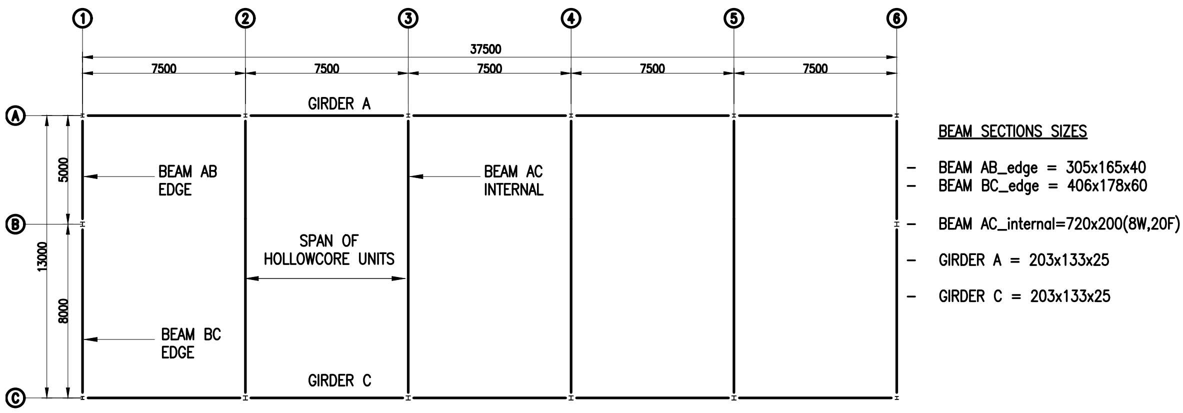


Figure 3.10: Long span floor layout in steel building supporting hollowcore units

3.7.2 Composite vs non-composite beam design

As discussed previously in Section 2.4.1.2, steel beams supporting hollowcore slabs can be designed using either composite or non-composite construction. Composite construction adds some complexity to the design stage but can result in significant cost savings. Small cost savings are achieved through a reduction in the amount of prestress required in the hollowcore slab. However, the main area for savings is with regard to the mass of steel beams that are required.

There is currently very limited information in South African design codes relating to the design of composite beams supporting hollowcore units. The design was therefore done in accordance with a design guide published by The Steel Construction Institute on the design of composite beams using precast concrete slabs, SCI P287 (Hicks and Lawson, 2003).

In order to investigate the potential mass savings that can be achieved through the use of composite construction, Table 3.3 compares several internal beams when designed using either composite or non-composite construction. Due to practical considerations, all edge beams were designed as being non-composite. This is because in order for edge beams to achieve composite action Hicks and Lawson (2003) specify that a minimum flange width of 230 mm is required. Therefore, edge beams are often designed as non-composite so that a similar section size may be used to the internal (composite) beams.

Table 3.3: Beam sizes required for composite and non-composite construction

Beam index	Non-composite	Composite	Mass difference [%]
5m internal	406x178x54	356x171x45	16.67%
8m internal	533x210x92	406x178x60	34.78%
13m internal	920x300 (8W,20F)	720x200 (8W,20F)	28.86%

From Table 3.3 it is clear that using composite construction results in significant reductions in the mass of steel required for the floor beams. This is clearly advantageous from a cost perspective and will greatly reduce the material cost of the steel beams. Furthermore, these material savings are compounded due to the repetitive nature of the structure. The only detailing requirements that need to be adhered to in order to achieve composite action involves welding shear studs onto the flanges of the beams, and placing transverse reinforcement into the hollowcore slab. For all composite beams a single shear stud in the centre of the beam's flange, at a spacing of 200 mm was used. In addition to this, 16 mm diameter reinforcement also at a spacing of 200 mm was calculated to be sufficient. The reinforcement is required to extend at least 500 mm into each hollowcore unit. An example of the calculation procedure that was followed for a composite steel beam supporting hollowcore slabs has been included in Appendix A.

3.7.3 Floor vibrations

The fundamental frequency of the composite steel beams was calculated to ensure that the vibrations under human induced loading would be of an acceptable level. The layout of the composite beams in the floor is such that the beams supporting hollowcore units frame directly into columns. Hicks and Lawson (2003) specify that if this is the case, then only the secondary beam mode need be considered. The fundamental frequency for a composite beam can be calculated by using the following formula:

$$fundamental frequency\ (f_n) = \frac{18}{\sqrt{\Delta}} \tag{3.2}$$

The deflection of a composite beam was calculated using the combined properties of the steel beam and hollowcore. For floors subjected to walking traffic, it is recommended that the fundamental frequency must be at least 3.55 Hz (Hicks and Lawson, 2003). The fundamental frequencies of the different beams in the floor system are given in Table 3.4.

Table 3.4: Fundamental frequency of beams in steel framed building with hollowcore floor slabs

Beam index	Fundamental frequency [Hz]
5m composite	11.08
8m composite	5.32
13m composite	3.89
5m edge	5.56
8m edge	3.44

From Table 3.4 it can be seen that all of the composite beams were found to be able to satisfy vibration criteria without any changes to normal design procedures. The fundamental frequency of the non-composite 8 m edge beam was marginally below the recommended limit, however by changing the section size from a 406 x 178 x 60 to a 406 x 178 x 67 the vibration criteria was satisfied.

3.7.4 Fire resistance of steel hollowcore floor system

The fire design methodology for the steel hollowcore structure is much simpler than the procedure used for composite metal deck floor structure. For the steel hollowcore structure, no secondary beams were required and consequently all beams required fire protection. Fire protection was supplied through the on-site application of vermiculite spray. The cost of providing the fire protection to the beams and columns is presented and discussed in more detail in Section 5.1.2.2.

In order to determine the fire resistance of the building, the fire resistance of the supporting steel beams and hollowcore units were considered in isolation. The component with the lowest fire resistance would thus define the fire resistance of the whole system. The fire resistance of each of these elements was calculated as follows:

- **Hollowcore units** - The prestressed hollowcore units used in the study have a standard fire rating of 1 hour (Concrete Manufacturers Association, 2011). If required, higher fire ratings can be achieved through the use of a structural topping, but a 1 hour rating is sufficient for the building under consideration.
- **Steel beams** - All steel beams receive fire protection through the on-site application of vermiculite spray. The methodology for specifying the fire protection thickness was discussed in Section 3.6.5.
- **Detailing** - In addition to the hollowcore slab and fire protection being provided to the steel beam, the transverse reinforcement must extend a distance of 600 mm into the hollowcore slab to achieve a 1 hour fire rating (Hicks and Lawson, 2003), see Figure 3.11. Transverse reinforcement was already required to extend a distance of at least 500 mm into the hollowcore slab for composite action to be achieved. Therefore having to increase the length of the reinforcement by 100 mm will have a negligible cost impact.

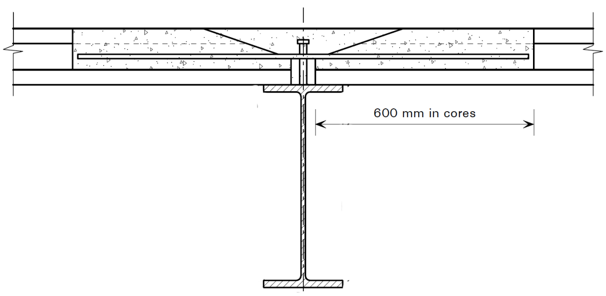


Figure 3.11: Detailing requirements for transverse reinforcement for a 60 minute fire rating (Hicks and Lawson, 2003)

3.8 Reinforced concrete frame with flat slab floor

The first concrete framed structural alternative that was considered was a reinforced concrete flat slab building. The floor layout and additional information regarding the structure is shown in Figure 3.12 below.

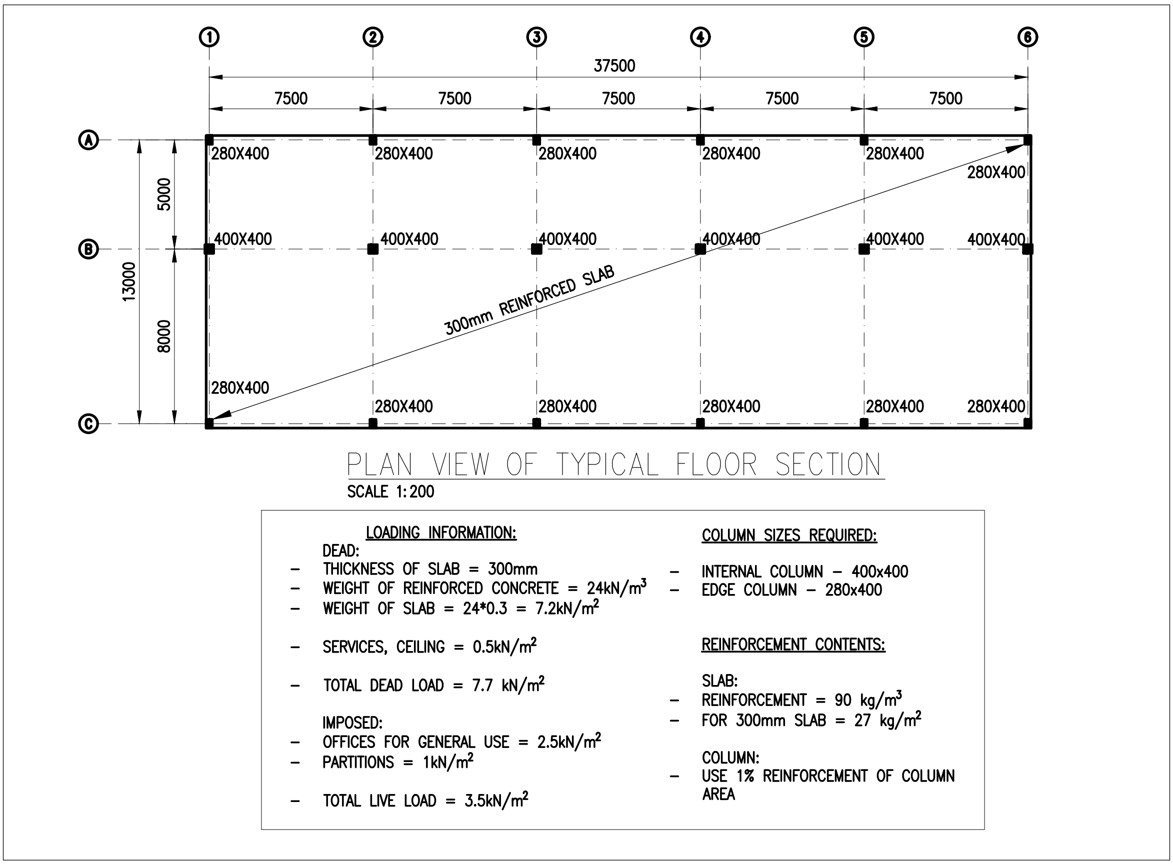


Figure 3.12: Plan view and information for flat slab structure

The loading information, in addition to several structural details, is shown in Figure 3.12, together with properties such as the slab thickness, column dimensions and reinforcement contents. The values and dimensions that were used were based on the consulting engineer's experience in the design of multi-storey commercial concrete structures. These values were checked with simple hand calculations to ensure that they were reasonable and realistic. In addition, the values were checked using a design guide published by the British Cement Association entitled Economic Concrete Frame Elements (Goodchild, 1997). An example of a section from the design guide that was used when comparing the values has been included in Appendix A.

The values that were obtained from the design guide were compared to the values that were recommended by the consulting engineer, and the results of this comparison are shown in Table 3.5. From Table 3.5 it can be seen that the values obtained from the design guide were very close to those from the experienced consulting engineer. By comparing the results of the hand calculations with the information obtained from the design guide, it is clear that the values that were recommended were realistic and suitable for the structure under consideration.

Some additional structural details associated with the reinforced concrete flat slab structure are discussed below:

- **Material properties** - All concrete used for slabs and columns had a 28 day strength of 30 MPa. All reinforcement used in the structure was high strength reinforcement with a

Table 3.5: Comparison of values obtained versus values from design guide

Parameter	Consulting engineer	Design guide
Slab thickness [mm]	300	283
Slab reinforcement content [kg / m^2]	27	23.2
Internal column sizes [mm x mm]	400 x 400	380 x 380

yield stress of 450 MPa.

- **Edge beams** - Edge beams are required in order to support the load from the brick and mortar cladding.
- **Fire protection** - The specifications of SANS 10100-1 (*The Structural Use of Concrete, Part 1: Design* 2000) Part 7, Table 43-46, regarding fire protection requirements were adhered to. The requirements for a 1 hour fire rating involved specifying a minimum cover for beams and floor slabs, in addition to adhering to minimum dimensions for the beam widths and slab thicknesses. As these requirements were adhered to, no additional fire protection was required.
- **Floor vibrations** - For the majority of building uses, concrete buildings are able to satisfy vibration criteria without any changes to normal design procedures (The Concrete Centre, 2006). Therefore, for the office building in the study, where no specialist vibration requirements are required, it was assumed that floor vibration criteria would be satisfied with no changes to normal design procedures.

3.9 Reinforced concrete frame with post-tensioned flat slab floor

The second concrete structural alternative that was considered was a reinforced concrete frame building with post-tensioned floor slabs. The floor layout and additional information regarding the structure is shown in Figure 3.13.

The loading information, in addition to several structural details, are shown in Figure 3.13, together with properties such as the slab thickness, column dimensions, reinforcement and cable contents. As for the RC flat slab structure, the values and dimensions that were used were based on the consulting engineer's experience. The values were again checked via hand calculations, and with the same design guide that was used for the RC flat slab structure (Goodchild, 1997). An example of a section from the design guide that was used when comparing the values has been included in Appendix A.

The values that were obtained from the design guide were compared to the values that were recommended from the consulting engineer, and this is shown in Table 3.6. From Table 3.6 it can be seen that the values obtained from the design guide were again close to the values

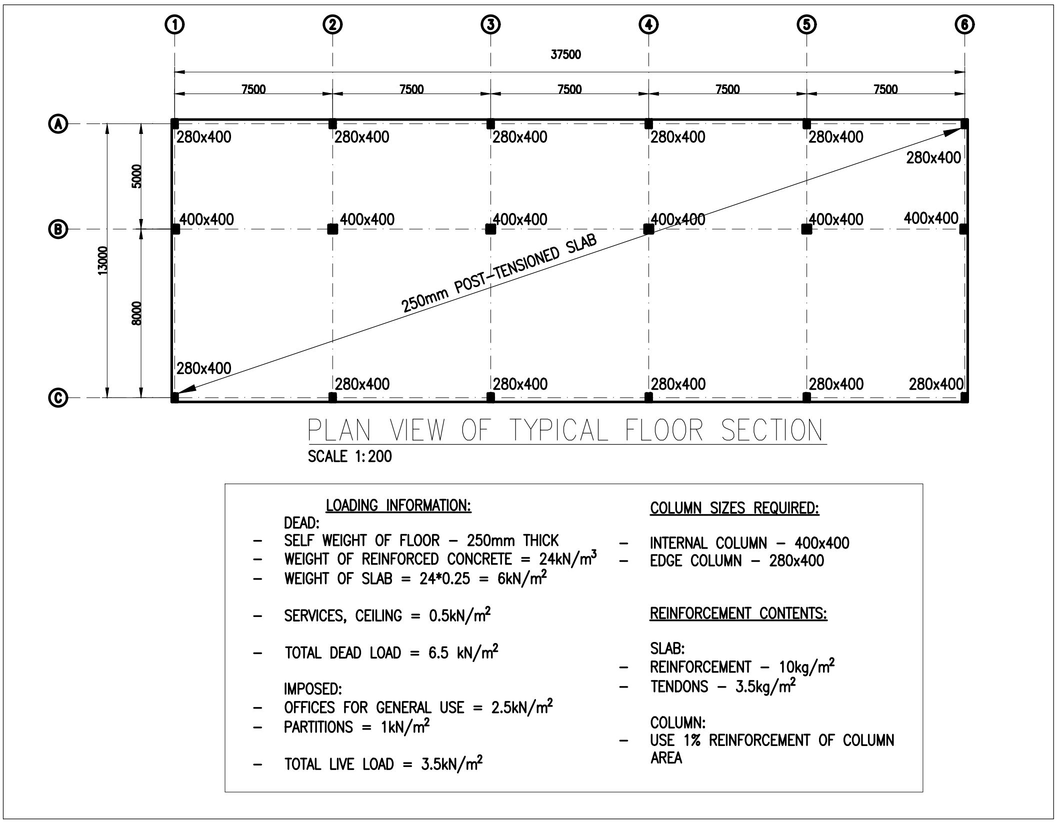


Figure 3.13: Plan view and information for post-tensioned concrete structure

recommended from the consulting engineer. Comparing the results of the hand calculations, and the information that was obtained from the design guide, it was clear that the values that were recommended were realistic and suitable for the structure under consideration.

Table 3.6: Comparison of values obtained versus values from design guide

Parameter	Consulting engineer	Design guide
Slab thickness [mm]	250	211.2
Slab reinforcement content [kg / m^2]	10	13
Slab cable content [kg / m^2]	14	10
Internal column sizes [mm x mm]	400 x 400	390 x 390

Some additional structural details associated with the post-tensioned concrete flat slab structure are discussed below:

- **Material properties** - All concrete used for slabs and columns had a 28 day strength of 30 MPa. The reinforcement used was high strength reinforcement with a yield stress of 450 MPa, while the cables that were used were unbonded, 15.2 mm diameter high strength cables with a yield stress of 1860 MPa.

- **Edge beams** - Edge beams were required in order to support the load from the brick and mortar cladding.
- **Fire protection** - The specifications of SANS 10100-1 (*The Structural Use of Concrete, Part 1: Design* 2000) Part 7, Table 43-46, regarding fire protection requirements were adhered to. The requirements for prestressed concrete slabs are similar to reinforced concrete floors with the only difference being a marginal increase in the thickness of the cover provided.
- **Floor vibrations** - For the majority of building uses, concrete buildings are able to satisfy vibration criteria without any changes to normal design procedures (The Concrete Centre, 2006). Therefore, for the office building in this study, where no specialist vibration requirements were required, it was assumed that floor vibration criteria was satisfied with no changes to normal design procedures.

3.10 Comparison of foundation sizes

As was mentioned in Section 3.4.1, the foundations were considered in detail for each of the structural alternatives. This was due to the fact that the different structural alternatives had varying self-weights, which therefore influenced the size of the foundations that were required. This section presents design results of the pad footings for each of the structural alternatives that were considered. The size of the pad footings were calculated using the base designer section in Prokon. An example of the Prokon output that was used to determine the size of the bases has been included in Appendix A.

The size of the foundations for each of the short and long span structural alternatives are shown in Table 3.7, as well as the total volume of concrete that is required in the footings. The top of all foundations were assumed to be located at a depth of 1 m below ground level, while the permissible bearing pressure of the soil was assumed to be 200 kPa. 30 MPa concrete was used as well as high strength reinforcement.

Table 3.7 shows that the steel framed structures required significantly smaller foundations compared to the concrete framed structures. Due to its relatively low self-weight, the short span steel composite structure required the smallest pad footings. The RC flat slab structure required the largest foundations, and the total volume of concrete required for its pad footings is approximately 95 % more than the total volume of concrete required for the short span composite structure. The cost implications that can be associated with the different foundation sizes are presented in Section 5.1.1.

Table 3.7: Summary of foundation sizes for various steel and concrete structural alternatives (length x breadth x height [m])

	SHORT SPAN STRUCTURES				LONG SPAN STRUCTURES	
Foundation index	RC flat slab	PT flat slab	Steel composite	Steel hollowcore	Steel composite	Steel hollowcore
A 1/6	1.9x1.5x0.4	1.9x1.4x0.4	1.5x1.1x0.3	1.6x1.2x0.3	2.1x1.6x0.4	1.6x1.2x0.3
B 1/6	2.7x2.1x0.5	2.5x1.9x0.5	2.1x1.6x0.4	2.2x1.7x0.4	1.3x1.0x0.3	2.0x1.4x0.4
C 1/6	2.7x2.1x0.5	2.5x1.9x0.5	2.1x1.6x0.4	2.2x1.7x0.4	2.1x1.6x0.4	2.3x1.7x0.5
A 2/3/4/5	2.7x2.1x0.5	2.5x1.9x0.5	2.1x1.6x0.4	2.2x1.7x0.4	3.1x2.4x0.6	3.3x2.5x0.6
B 2/3/4/5	3.6x2.8x0.7	3.4x2.4x0.7	2.9x2.2x0.6	3.1x2.4x0.6	-	-
C 2/3/4/5	3.1x2.4x0.6	3.0x2.3x0.6	2.5x1.9x0.5	2.7x2.0x0.5	3.1x2.4x0.6	3.3x2.5x0.6
Total volume of concrete [m^3]	71.04	60.54	36.55	41.78	41.87	46.90
Percentage difference [%]	94.34%	65.61%	0.00%	14.29%	14.54%	28.31%

Chapter 4

Construction programme comparison

Shorter construction time spans is arguably the most important benefit offered by steel construction (Steel Alliance, 2010). This reduction in construction time can be attributed to the fact that steel construction makes use of pre-fabricated components that can be rapidly assembled on site, which allows for shorter construction programmes and early access for following trades. For a steel framed structure, the construction time of the primary frame and floors has been shown to be as much as 40 % faster than for a reinforced concrete structure. This can result in time savings of up to 20 % of the total construction time (Steel Alliance, 2010). The shorter frame construction time allows a water-tight building envelope to be established early in the construction programme, which results in substantial programme benefits.

A reduction in construction time leads to a number of time-related savings. Examples of these savings include lower preliminary and general (P&G) costs, early occupation which leads to an earlier return on investment and reduced interest charges on borrowed capital. According to Davison (2012), time-related costs can amount to 3 - 5 % of the total project cost. Considering the fact that the a building's frame typically only accounts for 10 % of the total project cost (Steel Alliance, 2010), time-related savings can have significant cost implications. It is therefore important that these costs are considered when developing cost comparisons between different structural alternatives.

The purpose of this chapter is to reveal the differences in construction time that can be expected between the steel and concrete structural alternatives. The cost implications of these time differences were considered when developing the cost comparison in Chapter 5. This chapter therefore presents the construction programme for each of the steel and concrete structural alternatives that were considered in this study. Information is provided describing how the construction programmes were developed and the factors that were considered during the development of these programmes.

4.1 Development of construction programme

The following construction phases were identified to be of importance when developing the construction programme in this study:

1. Foundations and building substructure
2. Building frame and floors
3. Roof
4. Cladding
5. Services and finishes

In order to ensure that the construction programmes were an accurate reflection of the current practices in the South African construction industry, several meetings were held with industry professionals. The professionals who offered their expertise included building contractors, project management specialists in construction, and the CEO of a steel fabrication company. These meetings ensured that the construction programmes that have been developed represent construction speeds that are currently achieved on a regular basis in South Africa.

4.1.1 Construction programme information for all structures

This section provides information regarding the development of the construction programme for both the steel and concrete structural alternatives considered in the study:

- The time required for the construction of reinforced concrete pad footings was assumed to be the same for both the steel and concrete structural alternatives. Although the steel structures require smaller footings, the potential for time saving compared to the concrete structures was calculated to only be approximately one day. It was therefore decided to use the same construction time for the pad footings for all of the structural alternatives under consideration. Furthermore, while it is likely that the concrete framed structures would require a strip footing along the building's perimeter to support the masonry walls, the construction of such a foundation was not considered when developing the construction programmes.

- The time required to install the non-structural components has been accepted as being constant for all structural alternatives in the study. The non-structural components include the roof, cladding, services and finishes. The cladding, services and finishes will commence at the ground level and move upwards towards the roof until completion. Approximately 27 days are required per storey to erect cladding, and 36 days per storey to install services and apply finishes.

- Although the time required for the installation of the non-structural components is constant for all of the structural alternatives, key interfaces with preceding and following trades were considered.

- There is a single mobile crane located on the site which is able to service the entire project area. The crane is used for both the steel and concrete alternatives.

- The duration of the construction programmes were calculated assuming a five day work cycle with no work occurring on weekends.

4.1.2 Construction programme information for steel structures

Information is provided in this section regarding the development of the steel structural alternatives considered in this study:

- The use of an erection rate of approximately 25 steel members per day was recommended by the CEO of Union Steel as being realistic for the construction of a project of this size in South Africa. This rate was confirmed by Brown et al. (2009) who specified an erection rate of 20 - 30 pieces per day as being achievable. This erection rate was applied with regards to the erection of columns, beams and bracing members. Erection of the steel frame begins at gridline 1A, see Figure 3.1, where lateral bracing is present in both directions. This ensures that the structure will be stable during erection of the frame.
- Steel columns are erected in 2-storey lengths.
- All beam-to-column and beam-to-beam connections, column splices, as well as the attachment of bracing members to gusset plates are in the form of bolted connections.
- Fire protection was provided through the on-site application of vermiculite spray. The application of the vermiculite spray commences at ground level and progresses upwards towards the roof. According to a fire protection specialist, the vermiculite spray can be applied at a similar rate to the erection of the steel frame.
- Shear studs are to be welded at the steel fabricator's workshop, prior to the beams arriving on site.
- Both of the steel framed structures were designed using unpropped construction. The only propping that was required in some cases was temporary restraint at midspan of the beams during the construction condition. This restraint is to provide stability during construction and can be removed shortly after the floor system has been placed.
- The recommended lead-in time for structural steelwork was taken as six weeks. This allowed time for the submission and approval of drawings as well as the ordering, fabrication and delivery of steelwork to site. This lead-in time assumes that once the construction of the frame begins, sufficient steel will be available that the erection of the frame can continue to completion without having to stop.
- Steel columns are attached to a 1.5 m stub column extending from the base of the pad foundation. The concrete is given seven days to cure prior to the attachment of the steel column. The steel column is attached to a base plate and connected to the column plinth with holding down bolts.

4.1.3 Construction programme information for concrete structures

- The lead-in time for the concrete framed structures was taken as four weeks. This provides sufficient time for the preparation of bending schedules and the ordering of reinforcement. The shorter lead-in time enables construction of the frame to commence as soon as the foundations are completed.
- Construction of the frame begins at ground level and moves up until the roof. Floors are propped until such a time that the concrete has attained sufficient strength and thereafter the props are removed.

4.2 Construction programme comparison

Figure 4.1 provides a comparison of the construction programmes for each of the structural alternatives that were considered in this study. The programmes in Figure 4.1 provide a summary of the detailed construction programme for each of the structures. An example of a detailed construction programme for both a steel and a concrete structural alternative, revealing all aspects that were considered, has been included in Appendix B.

4.3 Discussion of construction programmes

4.3.1 Comparison of steel and concrete framed structures

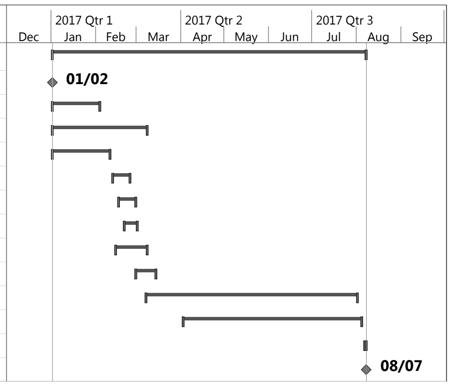
Considering the construction programmes shown in Figure 4.1, it can be seen that the steel framed structures offer a one month shorter construction period compared to the reinforced concrete structural alternatives. This time difference can primarily be attributed to the reduction in frame construction time between the steel and concrete structures. In addition, the use of unpropped construction for the steel framed structures enable following trades, such as cladding, finishes and services, to be incorporated early into the construction programme.

4.3.2 Comparison between steel framed structures

The duration of the construction programmes for the steel framed structural alternatives were very similar. Some of the differences between the construction programmes of the steel framed structures were the following:

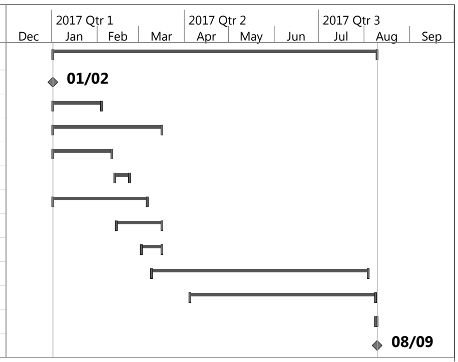
- **Steel frame** - The hollowcore structure has 42 % fewer floor beams compared to the composite structure, which results in a difference of 60 beams. Considering an erection rate of 25 steel members per day, this translates to a time saving of between two to three days.

ID	Task Name	Duration	Start	Finish
1	**STEEL COMPOSITE STRUCTURE**	**156 days**	**Mon 17/01/02**	**Mon 17/08/07**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
19	**Steel frame and composite floors**	**48 days**	**Mon 17/01/02**	**Wed 17/03/08**
20	**Lead in time for steel work**	**30 days**	**Mon 17/01/02**	**Fri 17/02/10**
24	**Erect steel frame**	**10 days**	**Mon 17/02/13**	**Fri 17/02/24**
30	**Bondek sheet and place reinforcement**	**8 days**	**Fri 17/02/17**	**Tue 17/02/28**
34	**Concrete pours**	**7 days**	**Tue 17/02/21**	**Wed 17/03/01**
38	**On-site application of vermiculite spray**	**16 days**	**Wed 17/02/15**	**Wed 17/03/08**
46	**Roof works**	**10 days**	**Wed 17/03/01**	**Tue 17/03/14**
49	**Cladding**	**105 days**	**Wed 17/03/08**	**Tue 17/08/01**
67	**Services and finishes**	**90 days**	**Mon 17/04/03**	**Fri 17/08/04**
120	**Close out Phase**	**1 day**	**Mon 17/08/07**	**Mon 17/08/07**
122	Final handover	0 days	Mon 17/08/07	Mon 17/08/07



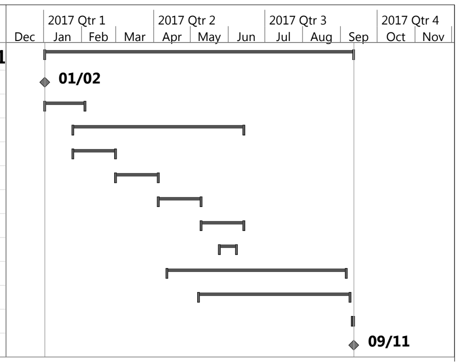
(a) Steel composite

ID	Task Name	Duration	Start	Finish
1	**STEEL HOLLOWCORE STRUCTURE**	**158 days**	**Mon 17/01/02**	**Wed 17/08/09**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
19	**Steel frame and hollowcore slabs**	**54 days**	**Mon 17/01/02**	**Thu 17/03/16**
20	**Lead in time for steel work**	**30 days**	**Mon 17/01/02**	**Fri 17/02/10**
24	**Erect steel frame**	**8 days**	**Mon 17/02/13**	**Wed 17/02/22**
30	**Hollowcore floor units**	**46 days**	**Mon 17/01/02**	**Mon 17/03/06**
38	**Onsite application of vermiculite spray**	**23 days**	**Tue 17/02/14**	**Thu 17/03/16**
46	**Roof works**	**10 days**	**Fri 17/03/03**	**Thu 17/03/16**
49	**Cladding**	**105 days**	**Fri 17/03/10**	**Thu 17/08/03**
67	**Services and finishes**	**90 days**	**Wed 17/04/05**	**Tue 17/08/08**
120	**Close out Phase**	**1 day**	**Wed 17/08/09**	**Wed 17/08/09**
122	Final handover	0 days	Wed 17/08/09	Wed 17/08/09



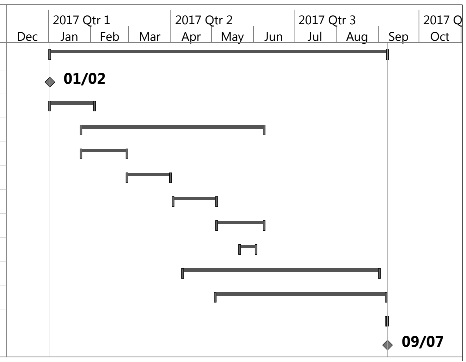
(b) Steel hollowcore

ID	Task Name	Duration	Start	Finish
1	**RC FLAT SLAB STRUCTURE**	**181 days**	**Mon 17/01/02**	**Mon 17/09/11**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
18	**Reinforced concrete frame**	**100 days**	**Wed 17/01/25**	**Tue 17/06/13**
19	**Ground floor columns and first floor slab**	**25 days**	**Wed 17/01/25**	**Tue 17/02/28**
28	**First floor columns and second floor slab**	**25 days**	**Wed 17/03/01**	**Tue 17/04/04**
37	**Second floor columns and third floor slab**	**25 days**	**Wed 17/04/05**	**Tue 17/05/09**
46	**Third floor columns to roof only**	**25 days**	**Wed 17/05/10**	**Tue 17/06/13**
51	**Roof works**	**10 days**	**Thu 17/05/25**	**Wed 17/06/07**
54	**Cladding**	**105 days**	**Wed 17/04/12**	**Tue 17/09/05**
72	**Services and finishes**	**90 days**	**Mon 17/05/08**	**Fri 17/09/08**
125	**Close out Phase**	**1 day**	**Mon 17/09/11**	**Mon 17/09/11**
127	Final handover	0 days	Mon 17/09/11	Mon 17/09/11



(c) Reinforced concrete flat slab

ID	Task Name	Duration	Start	Finish
1	**POST-TENSIONED STRUCTURE**	**179 days**	**Mon 17/01/02**	**Thu 17/09/07**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
18	**RC frame with post-tensioned floors**	**97 days**	**Wed 17/01/25**	**Thu 17/06/08**
19	**Ground floor and first floor slab**	**24 days**	**Wed 17/01/25**	**Mon 17/02/27**
29	**First floor to second floor**	**24 days**	**Tue 17/02/28**	**Fri 17/03/31**
39	**Second floor to third floor**	**24 days**	**Mon 17/04/03**	**Thu 17/05/04**
49	**Third floor to roof**	**25 days**	**Fri 17/05/05**	**Thu 17/06/08**
54	**Roof works**	**10 days**	**Mon 17/05/22**	**Fri 17/06/02**
57	**Cladding**	**105 days**	**Mon 17/04/10**	**Fri 17/09/01**
75	**Services and finishes**	**90 days**	**Thu 17/05/04**	**Wed 17/09/06**
128	**Close out Phase**	**1 day**	**Thu 17/09/07**	**Thu 17/09/07**
130	Final handover	0 days	Thu 17/09/07	Thu 17/09/07



(d) Post-tensioned flat slab

Figure 4.1: Summarised construction programmes for the various structural alternatives

- **Floor system** - The erection rates in steel framed structures are dominated by "hook time", which refers to the length of time members are connected to the crane (Brown et al., 2009). Reducing the number of crane lifts required during erection will therefore reduce the construction time. Bond-Dek sheets are able to be lifted in bundles and spread across the floor by hand, whereas hollowcore floor units need to be lifted and placed individually. In addition, the installation of the Bond-Dek sheets provides an immediate work platform, which allows following trades to begin.
- **Overall** - Ultimately, the advantages and disadvantages of the hollowcore and composite structure counteract one another, resulting in similar construction programmes for both the frame and overall construction time. The steel composite option however provides the fastest construction time by two days, due to the speed of erection and distribution that can be achieved when laying the Bondek metal deck.

4.3.2.1 Short and long span programme comparison

The construction programmes for the short and long span layouts were also considered. The long span layout requires fewer pad foundations, although they are slightly larger than those required for the short span alternatives. The construction time for the foundations can therefore be reduced by one day, i.e. from nine days to eight days. However, the lead-in time for the structural steelwork is longer than the construction time required for the foundations, which means that foundation construction is not a critical path activity, so no time savings are actually realised. The long span structural alternatives require fewer floor beams compared to the short span structures, which results in a time saving of two days. The non-structural components will be the same for both layouts. Overall, the time savings that can be achieved with the long span structure are approximately two days. This is a very small difference and, as such, the construction time was assumed to be the same as the short span structure when considering the time-related costs in this study.

4.3.3 Comparison between concrete framed structural alternatives

The duration of the construction programmes for both the concrete framed structural alternatives were found to be very similar. Some of the differences between the construction programmes of these structures were the following:

- **Frame construction** - The majority of the components of the frame construction are similar for the RC flat slab and the PT flat slab structures. Both structures require similar levels of propping, edge beams to support cladding loads, and the same column sizes. The main difference is the floor slab for each system. The PT flat slab structure benefits from a reduction in reinforcement being required, due to the presence of the tendons, and less concrete due to the thinner slab. There is however additional effort required to tension the tendons which is not required for the RC flat slab structure.

- **Overall** - Ultimately, the time differences between the concrete framed structures are very small. The PT flat slab structure offers a reduction in construction time of two days compared to the RC flat slab structure.

4.3.4 Additional comments

The following additional comments can be made regarding the construction programmes of the structures considered in this study.

- **Lead-in time** - The lead-in time is an important consideration when developing the construction programme in steel framed structures. It has been shown in this study that steel construction is able to offer significant advantages in frame construction time compared to concrete construction. However, these time advantages can only be realised if the steel is available, and all preparatory work has been completed to allow steel construction to commence. It has therefore been identified to be very important that there is early collaboration between the main contractor and the steel fabricator.

- **Project size** - For the structure considered in this study, the foundations were able to be constructed relatively quickly. Therefore, the lead in time resulted in a slight delay in the frame construction for the steel structural alternatives. In a larger project, more substantial foundations could be required, with basement levels possibly being necessary. This would serve to manage the lead-in time more efficiently, and time associated with the construction of the basements and foundations could be used for the procurement of the steelwork. Another aspect to consider is that the structure in this study had only three suspended floors, and it can be expected that a structure with more floors would lead to greater time differences between the steel and concrete structural alternatives.

Chapter 5

Cost comparison

This chapter presents the results of the cost comparison between the various structural alternatives that were considered in this study. In order to provide a realistic cost comparison, all key aspects influencing the building cost were considered. The two main areas that were identified to be of importance when developing the cost comparison were the total capital investment, and all time-related costs. The consideration of both of these cost aspects allowed for a holistic view of the actual cost for each of the structural alternatives.

The building costs and rates used in this section were obtained from De Leeuw Quantity Surveyors in Stellenbosch. The rates were sourced from a variety of recently tendered projects, which were of a similar nature to the structure considered in this study. This ensured that the costs were realistic and up to date. The rates are a reflection of typical construction costs in the Western Cape province of South Africa in 2016. Structural steel rates obtained from these projects were discussed with the CEO of Union Steel to ensure that they were in line with the costs that could be expected in a project of this nature.

5.1 Total construction cost

The total construction cost consists of all costs associated with the construction of the building. This includes the cost of the raw materials, fabrication, erection, labour, formwork and propping, and any other costs associated with the construction of the building. Preliminary and general (P&G) costs would also typically be included in the total construction cost. However, because they depend on the duration of the construction programme, they have been included in Section 5.3 instead, which presents the time-related costs that were considered in this study. Capital costs, such as professional fees and land costs, were also not included in this section, and form part of the total capital investment, which is discussed in Section 5.2. The total construction cost was broken down into three main components in this study:

1. Foundations and building substructure
2. Building frame and floors
3. Non-structural components

5.1.1 Foundations and building substructure

The cost of the foundations and building substructure was considered in detail for each of the structural alternatives. Each structure has a different self weight, which in turn influenced the size of the foundations that were required. A summary of the different foundation sizes for each of the structural alternatives was shown in Section 3.10. As was discussed in Section 3.4.1, the cost implications associated with the construction of a strip footing for the concrete framed structures were not considered during this study. Therefore, the foundation and substructure costs included the reinforced concrete pad footings below column positions, and the reinforced concrete ground floor slab.

The cost of the foundations and building substructure for the various structural alternatives is presented in Table 5.1 and Figure 5.1. Additional information regarding the calculation of the foundation costs, and the rates that were used, can be found in Appendix C.

Table 5.1: Comparison of foundation and substructure costs for steel and concrete structural alternatives

COST COMPONENT	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
Excavation	R 88 369.38	R 80 837.13	R 63 022.17	R 62 847.64	R 68 175.80	R 67 456.63
Concrete	R 240 019.81	R 221 516.34	R 180 830.55	R 188 314.04	R 190 242.69	R 195 967.01
Reinforcement	R 71 329.56	R 63 609.12	R 46 038.58	R 49 569.52	R 49 876.75	R 52 778.90
Formwork	R 24 382.27	R 23 548.00	R 18 145.42	R 16 685.44	R 18 753.63	R 12 587.25
Total	R 424 101.02	R 389 510.60	R 308 036.71	R 317 416.64	R 327 048.87	R 328 789.79
% Difference	37.68%	26.45%	0.00%	3.05%	6.17%	6.74%

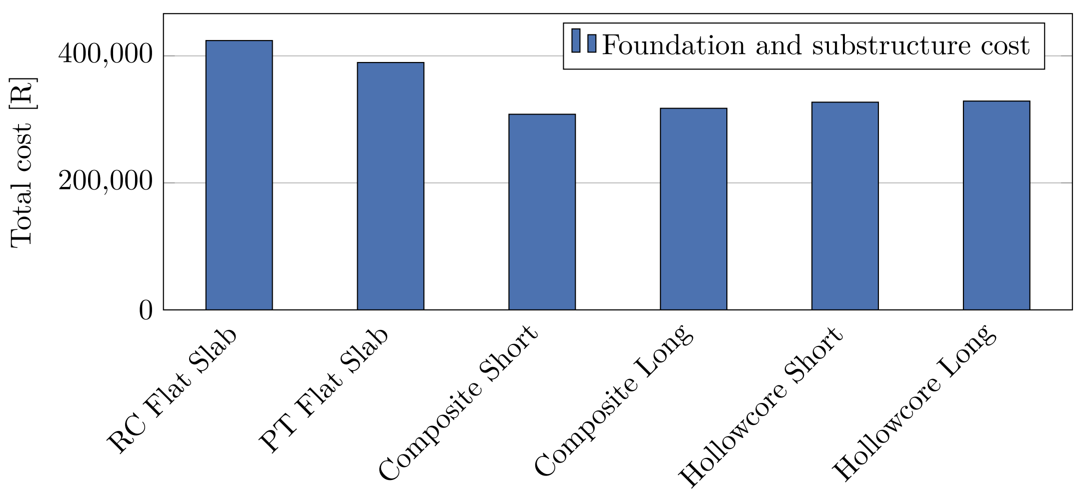


Figure 5.1: Summary of the total foundation and substructure cost for various steel and concrete structural alternatives

Table 5.1 and Figure 5.1 show that the short span steel composite structure offered the lowest foundation and substructure cost. This was to be expected, as this structure has the lowest self weight, which meant that it required the smallest foundations. The foundation costs for the steel hollowcore structures were found to be approximately 6 % higher than for the steel composite structure. This can be attributed to the slightly higher self weight of the hollowcore floor units compared to the Bond-Dek slab. Although the long span structures required fewer pad footings, due to the absence of the internal columns, the increased load on the columns meant that the foundation costs remained similar compared to the short span structural alternatives.

The concrete framed structures had significantly higher foundation and substructure costs compared to the steel framed structures. Again, this was to be expected because of the higher self weight of the reinforced concrete structures compared to the steel structures. Although it was not considered, the additional cost of providing the strip footing for the concrete framed structures would have contributed to increasing this cost difference. The total foundation and substructure cost for the reinforced concrete flat slab structure was found to be 38 % higher than the composite steel deck structure, while the post-tensioned flat slab foundation costs were 26 % higher.

5.1.2 Building frame and floors

The cost of the building frame was calculated for each of the steel and concrete structural alternatives. This is the primary area for potential cost differences between the buildings, with different structural systems being employed in each structure. The major cost components of the building's frame included the columns, beams, connections, floor slabs, fire protection, reinforcement, formwork and propping, and any other aspects making up the frame of the building. The cost of providing lateral stability was assumed to be the same for each of the steel and concrete structural alternatives and was therefore not included in the frame costs. The calculation of the fire protection costs for the steel framed structural alternatives is discussed in greater detail in Section 5.1.2.2. A more detailed breakdown of the frame cost calculations for each of the structural alternatives can be found in Appendix C. The frame costs for each of the steel and concrete structural alternatives are shown in Figure 5.2.

Figure 5.2 revealed that both concrete structural alternatives provided lower frame costs than the cheapest steel alternative, with the post-tensioned flat slab structure providing the lowest cost of all the options. The short span steel hollowcore structure has the lowest frame cost out of all the steel alternatives, while the long span composite structure has the highest frame cost. The short span hollowcore option was found to be 23 % more expensive than the post-tensioned flat slab structure, which equated to a cost difference of R 481 570.

The frame cost of the reinforced concrete flat slab structure was 11 % higher than the post-tensioned flat slab structure. The main reasons for this cost difference could be attributed to the reduction in slab reinforcement and thickness in the post-tensioned structure, due to the presence of the high strength steel tendons.

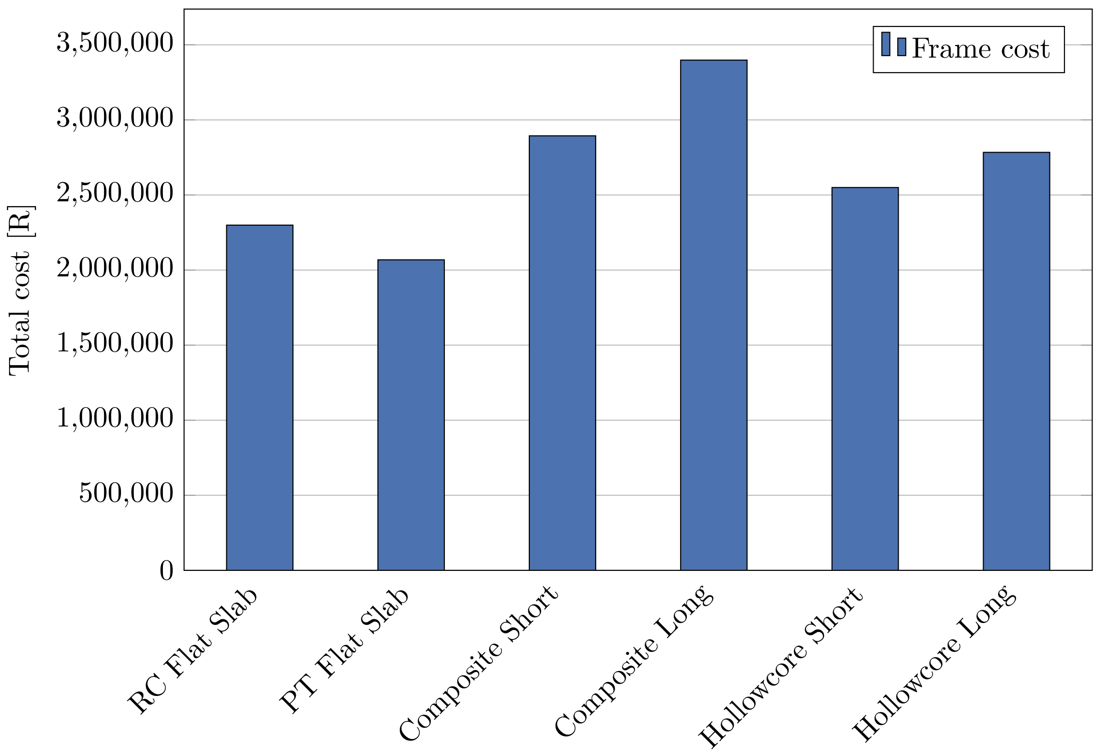


Figure 5.2: Summary of the total frame cost for the various steel and concrete structural alternatives

As was expected, the frame cost of the long span structures were higher than the short span structures for both the composite and hollowcore alternatives. This can mainly be attributed to the heavier steel sections that were required to span the full 13 m width of the building. Longer spanning structures do however provide the benefit of column free floor space, which allows greater flexibility in the building's intended use. This is clearly beneficial from a client's point of view, and could justify the increased construction cost. The long span composite structure had the highest frame cost, and was found to be approximately 65 % more costly than the post-tensioned flat slab structure. The frame cost of the long span hollowcore structure was approximately 18 % less than the long span composite structure, due to the absence of the heavy secondary beams which reduced the mass of steelwork required.

5.1.2.1 Breakdown of frame costs

Figure 5.3 shows a breakdown of the frame costs for each of the steel and concrete structural alternatives. The total frame cost was broken down into the cost of the floor system, columns, and any fire protection that was required.

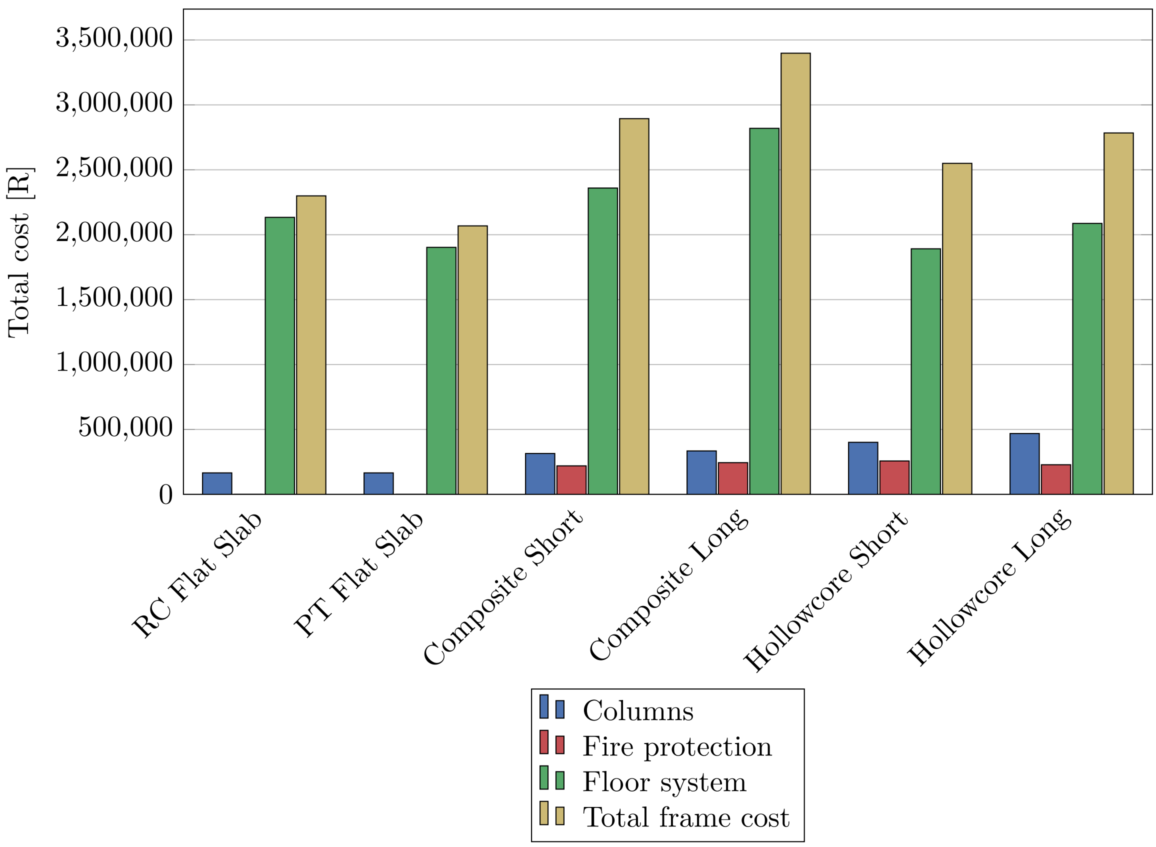


Figure 5.3: Breakdown of the total frame cost of each of the steel and concrete structural alternatives

Figure 5.3 shows that the cost of the floor system is similar between the concrete framed structures and the steel framed structure supporting hollowcore units. The concrete structures however, benefit from lower column costs and the fact that no additional fire protection is required, which results in a lower total frame cost. The cost of providing passive fire protection ranged between 8 - 10 % of the total frame cost for the steel framed options. The calculation of the fire protection costs is discussed in more detail in Section 5.1.2.2.

The steel composite structures clearly have the highest floor cost. This can be attributed to the large number of steel beams and shear connectors that were required in these structures, whereas the steel hollowcore structures required significantly fewer beams and shear connectors. Even when a partial shear connection is employed in the design of the composite beams, a large number of shear connectors were still required when all the beams are designed to act compositely, which contributes to the high floor cost.

5.1.2.2 Cost of fire protection for steel framed buildings

This section presents the costs of providing fire protection to the steel framed structures considered in this study. As was mentioned in Section 3.8 and 3.9, no additional fire protection was required for the concrete framed structures in this study, and they are therefore not discussed further in this section. The cost of providing fire protection typically ranges between 10 - 15 % of the total frame cost in commercial multi-storey steel buildings (Barrett Byrd Associates, 2016). Therefore, the selection of fire protection materials, and the fire design of multi-storey steel framed structures can have a significant influence on the total frame cost of the building.

The fire protection costs presented in this section are based on the on-site application of either intumescent paint or vermiculite spray. As mentioned in Section 3.6.5 the building considered in this study requires a one hour fire rating as specified in Table 5 of Part T in SANS 10400 (*The application of the National Building Regulations* 2008). Therefore, the fire protection costs for achieving a one hour fire rating were calculated. In addition, the costs for a two hour fire rating were also calculated. If the occupancy of the building were to change in the future, or if more stories were added, the required fire rating could potentially increase. It is therefore of value to know the fire protection costs for a higher fire rating. A summary of the fire protection costs for each of the steel structural alternatives are presented in Table 5.2 and Figure 5.4. Additional information regarding the calculation of these costs can be found in Appendix C.

Table 5.2: Summary of total fire protection costs for different steel structural alternatives

MATERIAL AND FIRE RATING	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
VERMICULITE 60 minutes	R 219 595.16	R 244 254.97	R 257 159.90	R 228 178.32
INTUMESCENT 60 minutes	R 326 520.22	R 343 891.60	R 362 716.88	R 326 128.82
VERMICULITE 120 minutes	R 314 570.01	R 324 149.27	R 353 949.69	R 340 289.72
INTUMESCENT 120 minutes	R 1 013 528.83	R 924 982.17	R 993 743.05	R 856 013.94

Table 5.2 and Figure 5.4 compare the cost of providing passive fire protection for various steel structural alternatives, when different fire ratings and fire protection materials were used. The costs of 13 mm thick plasterboard to board up the columns was included when vermiculite spray was used, due to the fact that vermiculite spray is not aesthetically pleasing and needs to be concealed from view. No additional costs were associated with concealing the floor beams if vermiculite spray is used, as all structural alternatives have a suspended ceiling which conceals the beams from sight. The cost of intumescent paint for a two hour fire rating includes the cost of filling the voids above the steel beam, which is a requirement specified by the ASFP Yellow Book 5th Edition (Association for Specialist Fire Protection, 2014). Considering the fire protection costs in Figure 5.4, some of the main conclusions that could be drawn were the following:

- Vermiculite spray provides the cheapest option for both the 60 and 120 minute fire rating.
- Intumescent paint is approximately 50 % more expensive than vermiculite spray when a

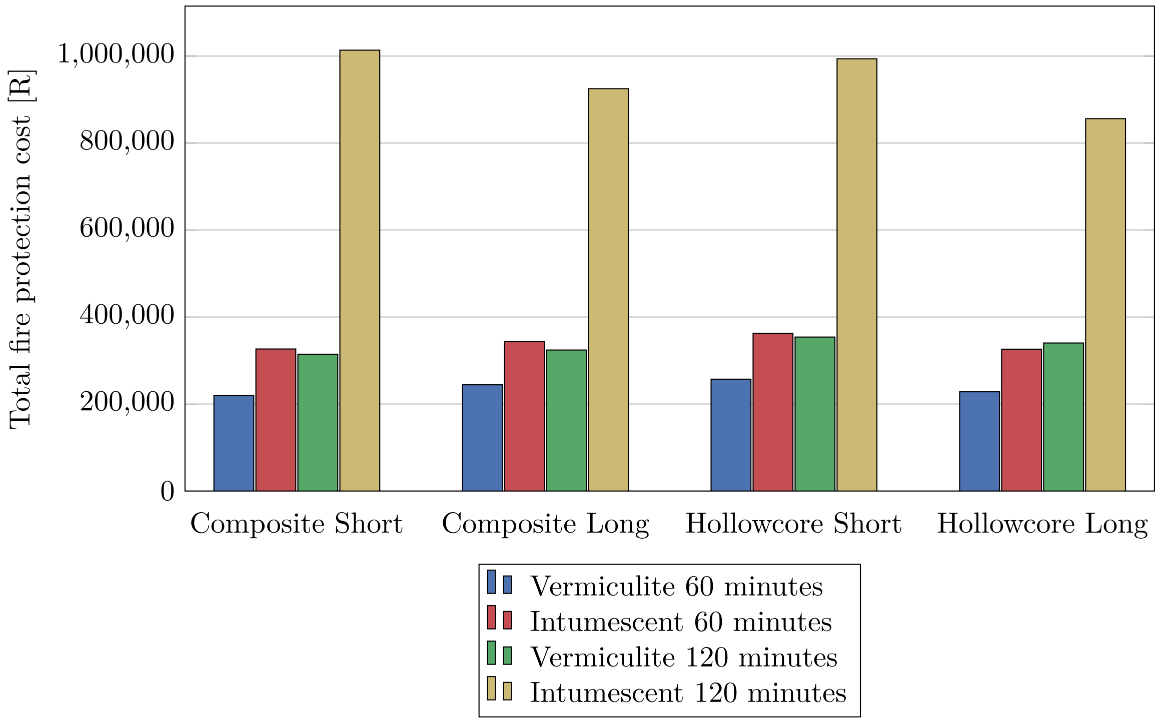


Figure 5.4: Summary of the total fire protection costs for various steel structural alternatives, materials and fire ratings

60 minute fire rating is required. It is however aesthetically pleasing, which means the steel can be left exposed.

- When vermiculite spray is used the cost increase associated with increasing the fire rating from a 60 to a 120 minute fire rating is approximately 45 %, but with intumescent paint a 120 minute fire rating costs between 2.5 to 3 times as much as a 60 minute rating. This significant increase in cost can be attributed to the large increase in the dry film thickness of intumescent paint required.

It can therefore be concluded that if a 60 minute fire rating is required the vermiculite spray offers the cheapest option. However, if an aesthetic finish is required, the additional cost of intumescent paint cost could be justified. For a 120 minute fire rating the cost of intumescent paint becomes very high and the most cost-effective solution would be to use vermiculite spray.

The fire protection costs for the composite steel deck structures that are shown in Table 5.2 were calculated based on the results of the Slab Panel Method (SPM), which was discussed in Section 3.6.4. The SPM design revealed that only the primary beams along the slab panel edges required fire protection, with all secondary beams being left unprotected. The division of the floor into the slab panels employed in this study was shown in Figure 3.7. Employing the SPM resulted in lower fire protection costs, which are shown in Table 5.3. The costs presented in Table 5.3 reflect the costs associated with providing fire protection to the floor beams only. All columns require fire protection, regardless of whether or not the slab panel method is used, and as such only the fire protection costs and savings associated with protecting the beams are shown.

Table 5.3: Reduction in fire protection costs through the implementation of Slab Panel Method

MATERIAL AND FIRE RATING	COMPOSITE SHORT				COMPOSITE LONG			
	NO SPM	SPM	DIFF [%]	TOTAL SAVING	NO SPM	SPM	DIFF [%]	TOTAL SAVING
VERMICULITE 60 minutes	R 92 010.62	R 44 079.42	52.09%	R 143 793.60	R 107 224.02	R 56 479.82	47.33%	R 152 232.60
INTUMESCENT 60 minutes	R 145 076.99	R 70 261.89	51.57%	R 224 445.30	R 154 772.97	R 82 077.53	46.97%	R 218 086.32
VERMICULITE 120 minutes	R 138 967.76	R 66 575.12	52.09%	R 217 177.92	R 142 919.53	R 75 902.19	46.89%	R 201 052.02
INTUMESCENT 120 minutes	R 435 486.20	R 214 531.85	50.74%	R 662 863.06	R 387 859.21	R 209 051.35	46.10%	R 536 423.59

Table 5.3 reveals that the use of the SPM results in substantial savings in fire protection costs. The cost savings that were realised for both the short span and long span composite structure were approximately 50 %. Significant cost savings were achieved with intumescent paint and vermiculite spray, for both a 60 and 120 minute fire rating. When vermiculite spray is used for a 60 minute fire rating, the total cost savings amount to approximately R 150 000.00. It is important to note however, that the building in this study has only three suspended floors. If the number of floors were to be increased, and potentially the required fire rating with it, the savings that would be achieved through the use of the SPM would also increase.

5.1.3 Cost of non-structural components

The final component of the total construction cost that was calculated was the cost of the non-structural components. These costs were considered to be the same for each of the steel and concrete alternatives considered in this study. The inclusion of these cost components is of value because it assisted in revealing what proportion of the total construction cost was contributed by the building's frame. If the cost of the building's frame only contributed a small percentage of the total construction cost, it could conceivably be more economical to pay more for the structural frame of the building, if it allowed for other advantages to be realised. An example of a potential advantage could be a shorter construction programme, which would allow income to be generated at an earlier stage, which could justify the increased frame cost.

In order to calculate the cost of the non-structural components, rates were obtained from De Leeuw Quantity Surveyors in Stellenbosch, who possess a great deal of experience in the costing of multi-storey commercial structures. They were thus able to provide rates that are applicable to a typical multi-storey office building in South Africa. Table 5.4 below shows the rates and quantities that were used to calculate the total cost of the non-structural components.

Table 5.4: Summary of quantities and costs for non-structural components

COST COMPONENT	UNIT	QUANTITY	RATE	TOTAL COST
Roof	m^2	487.5	R 1 375.00	R 670 312.50
Cladding	m^2	1515	R 1 350.00	R 2 045 250.00
Electrical services	m^2	1950	R 800.00	R 1 560 000.00
Mechanical services	m^2	1950	R 1 500.00	R 2 925 000.00
Plumbing and wetpoints	m^2	100	R 10 500.00	R 1 050 000.00
Finishes and fitments	m^2	1950	R 1 750.00	R 3 412 500.00
Lift	No.	1	R 500 000.00	R 500 000.00
Parking around building	m^2	1750	R 1 250.00	R 2 187 500.00
TOTAL COST OF NON-FRAME COMPONENTS				R 14 350 600.00

5.1.4 Total construction cost

The total construction cost could now be determined by summing the building foundation and substructure cost, the building frame cost and the cost of the non-structural components. The total construction cost for each of the structural alternatives is shown in Table 5.5 and Figure 5.5.

Table 5.5: Calculation of the total construction costs for each of the structural alternatives considered in the study

COST COMPONENT	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
Total foundation and substructure cost	R 424 101	R 389 510	R 308 036	R 317 416	R 327 048	R 328 789
Total frame cost (incl. fire protection)	R 2 299 171	R 2 068 015	R 2 893 967	R 3 398 058	R 2 549 585	R 2 783 779
Total cost of non-frame components	R 14 350 600	R 14 350 600	R 14 350 600	R 14 350 600	R 14 350 600	R 14 350 600
Total construction cost	R 17 073 880	R 16 808 130	R 17 552 610	R 18 066 080	R 17 227 240	R 17 463 170
Cost difference	R 265 750	R 0	R 744 480	R 1 257 950	R 419 110	R 655 040
Percentage difference	1.58%	0.00%	4.43%	7.48%	2.49%	3.90%

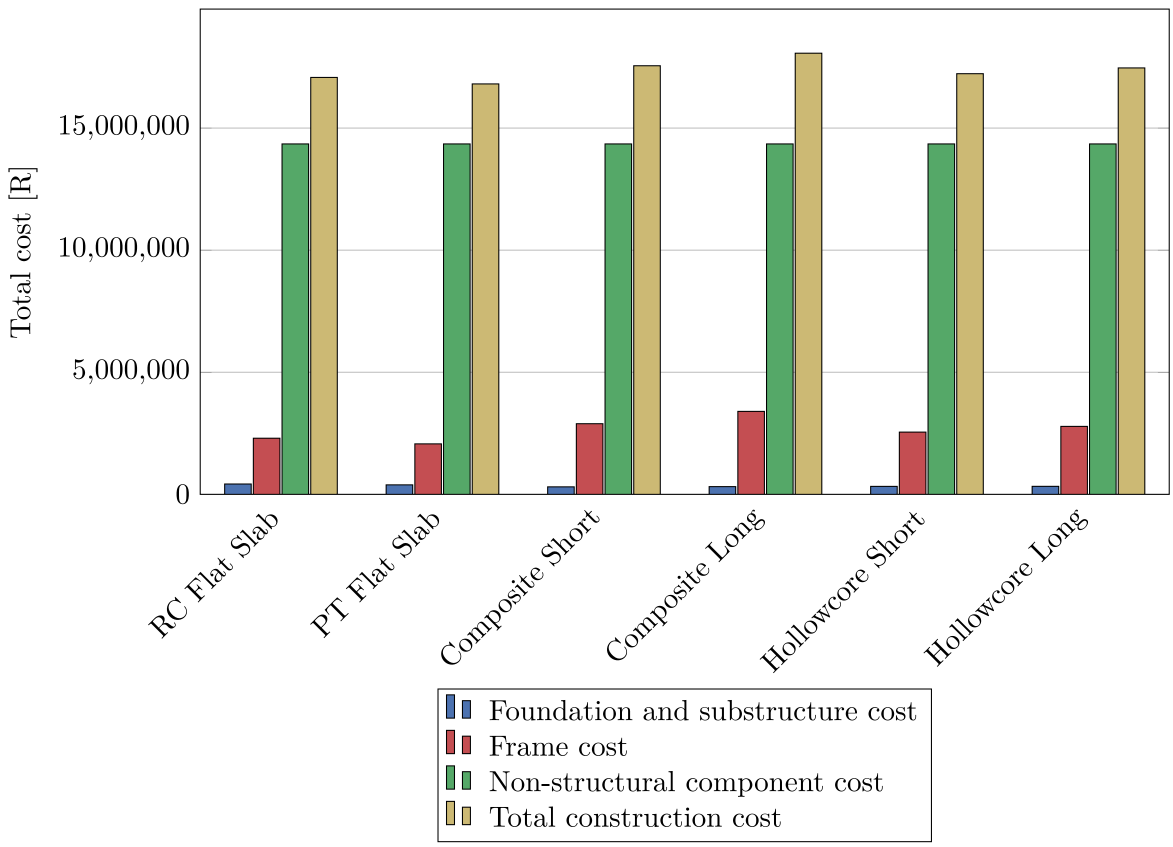


Figure 5.5: Comparison of the total construction cost for each of the steel and concrete structural alternatives

Table 5.5 and Figure 5.5 reveal that the concrete framed structures offered the lowest total construction costs, with the post-tensioned flat slab structure proving to be the cheapest option. The cheapest steel option was the short span hollowcore structure which was found to be approximately 2.5 % more costly than the PT flat slab structure. This difference equated to a cost difference of approximately R 400 000. The long span steel composite structure had the highest total construction cost, and was found to be approximately R 1 260 000 more costly than the PT flat slab structure.

However, it is important to note that no time-related costs have been considered at this stage of the comparison. Time-related costs are discussed in Section 5.3, and need to be considered before a statement regarding the cost-effectiveness of the various structural alternatives can be made.

5.2 Total capital investment

There are several costs that are incurred during a project which do not form part of the total construction cost. These costs are however important to consider because they are added to the construction cost, to obtain the total capital investment for the project. The total capital investment dictates the sum of money to be loaned from the bank in order to undertake the development. A number of additional cost components were considered, with the two major cost items in most development projects being the land cost and professional fees. Advice was obtained from De Leeuw Quantity Surveyors regarding typical values and rates for these components when used for typical office buildings in South Africa. Table 5.6 below indicates the additional cost components that were considered when calculating the total capital investment.

Table 5.6: Additional cost components considered

COST COMPONENT	TOTAL COST
Land costs - Building area + parking area	R2 722 600.00
Professional fees	15% of total construction cost
Disbursements	0.5% of total construction cost
Health and safety consultant	R 50 000.00
Local authority costs	R 150 000.00
Promotional costs	R 100 000.00
Development contingencies	0.5% of total capital investment

The additional cost components were calculated separately for each of the structural alternatives, due to the fact that a number of these costs were dependent on the construction cost. Table 5.7 below shows the total capital investment for each of the structural alternatives, excluding any time-related costs which are considered in the next section.

Table 5.7: Total capital investment for the various structural alternatives

COST COMPONENT	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
Total additional costs	R 6 125 900	R 6 077 840	R 6 141 940	R 6 232 750	R 6 084 400	R 6 126 120
Total capital investment (excl. time-related costs)	R 23 199 775	R 22 885 974	R 23 694 549	R 24 298 832	R 23 311 636	R 23 589 293
Percentage difference	1.37%	0.00%	3.53%	6.17%	1.86%	3.07%

5.3 Time-related costs

A shorter construction programme allows for a reduction in a number of time-related costs. Chapter 4 revealed that the steel structural alternatives offered a reduction in construction time of approximately one month, compared to the reinforced concrete structures. The cost implications that could be associated with the shorter construction programme are presented in this section. The primary time-related costs that were considered in this study included the following:

1. Preliminary and general (P&G) costs

2. Reduced interest costs

3. Earlier return on investment

When calculating the influence of the time-related costs it has been assumed that funds are mobilised and that payments take place at the beginning of each month. In addition, it was assumed that a tenant could be found to occupy the building immediately after construction was completed.

5.3.1 Preliminary and general (P&G) costs

Preliminary and general (P&G) costs represent a wide range of building costs that are incurred during a project. Some examples of cost aspects that are typically considered in this section are site establishment costs, crane hire, material control and storage costs, mobilisation cost and site supervision, to name a few. P&G costs vary depending on the scale of the project and typically contribute between 10 - 15 % of the total construction cost for an office building (Davison, 2012). The use of steel construction allows for a reduction in P&G costs primarily due to the shorter construction programme that can be achieved compared to concrete construction. Furthermore, steel construction is typically less site intensive than concrete construction, due to the fact that steel components are fabricated off-site, which assists in reducing the cost of P&G costs.

Therefore, due to the aspects mentioned above, the P&G costs were calculated separately for the steel and concrete structural alternatives in this study. De Leeuw Quantity Surveyors recommended that the P&G costs be calculated as a percentage of the total construction cost. The following percentages were recommended for a typical steel and concrete multi-storey office building:

- Concrete structural alternatives = 12.5 %

- Steel structural alternatives = 10 %

The P&G costs were calculated based on the recommended percentage, and the costs obtained are shown in Table 5.8 below.

Table 5.8 reveals that the steel framed structures experience significant savings in P&G costs, compared to the concrete framed structures. The difference in P&G cost proved to be approximately 20 %, which equated to a cost difference of approximately R 400 000.00. P&G costs are therefore an important consideration when developing cost comparisons, and the use of steel construction can result in significant savings in this area.

Table 5.8: P&G costs for each of the steel and concrete structural alternatives

	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
P&G costs	R 2 134 234	R 2 101 015	R 1 755 260	R 1 806 607	R 1 722 723	R 1 746 316
Cost difference	R 411 510	R 378 292	R 32 537	R 83 884	R 0	R 23 593
% Difference	23.89%	21.96%	1.89%	4.87%	0.00%	1.37%

5.3.2 Reduced interest costs

Finance costs in a project can be significant, particularly when borrowed money is used to undertake the project. A shorter construction period means that less interest is incurred on borrowed capital before an income can be generated. In order to calculate the interest incurred during the project a cash flow diagram was developed for each of the structural alternatives based on the construction programme developed in Chapter 4. An interest rate of 10.5 % per annum was recommended by De Leeuw Quantity Surveyors as the current interest rate for a development loan in South Africa in 2016. This annual interest rate was converted into an effective monthly rate and interest was calculated based on the development cash flow which is presented in Section 5.4. A comparison of the interest incurred for each of the structural alternatives during the construction period is shown in Table 5.9. From Table 5.9 it is clear that the steel framed structures benefit from the shorter construction programme and this results in interest savings in excess R 100 000.00 compared to the concrete framed structures.

Table 5.9: Interest incurred during construction for the various structural alternatives

COST COMPONENT	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
Interest incurred during construction period	R 1 073 693	R 1 060 400	R 954 860	R 977 083	R 940 778	R 950 989
Cost difference	R 132 915	R 119 622	R 14 082	R 36 305	R 0	R 10 211
Percentage difference	14.13%	12.72%	1.50%	3.86%	0.00%	1.09%

5.3.3 Earlier return on investment

A shorter construction programme means that income can be generated at an earlier stage, which allows for an earlier return on the capital investment in the project. Income for the structure was calculated based on the rentable floor area and number of parking bays that have been constructed. De Leeuw Quantity Surveyors and Isipani Construction recommended a rate R 150 / m^2 of floor area per month and R 100 per parking bay per month. Both of these rates were varied as part of the sensitivity analysis performed in Chapter 6. The total monthly income for the structure was calculated following the procedure shown in Table 5.10.

Table 5.10: Methodology for calculating building income

Net income after maintenance etc.	R 150 / m^2
Rentable floor area (Reduce gross area by 5 %)	1850 m^2
Total monthly income from office space rental	R 277 500
Net income from parking bay	R100/bay/month
Number of bays (4 bays per 100 m^2 of floor area)	70 bays
Total monthly income from parking rental	R 7 000
Total monthly income	R 284 500

5.4 Cash flow and overall cost effectiveness

The final step of the cost comparison was the calculation of the cash flow for each of the structural alternatives in this study. The cash flow incorporated all aspects that influenced the cost effectiveness of the structure. An example of how the cash flow was calculated is included in Appendix C. All construction and additional cost components were considered, as well as the time-related costs discussed in Section 5.3. The results of including the impact of the cash flow on the total cost of each of the structural alternatives are shown in Table 5.11 and Figure 5.6. The total cost of the structural alternatives are compared at the date when the construction of the concrete framed structures ends. For the purposes of this study it was assumed that the construction of all alternatives started simultaneously.

Table 5.11: Total capital investment for the various steel and concrete structural alternatives

COST COMPONENT	RC FLAT SLAB	PT FLAT SLAB	STEEL COMPOSITE SHORT SPAN	STEEL COMPOSITE LONG SPAN	STEEL HOLLOWCORE SHORT SPAN	STEEL HOLLOWCORE LONG SPAN
Total capital investment (incl. time-related costs)	R 26 407 700	R 26 047 381	R 26 332 703	R 27 016 032	R 25 899 696	R 26 213 681
Cost difference	R 508 003	R 147 684	R 433 006	R 1 116 336	R 0	R 313 984
Percentage difference	1.96%	0.57%	1.67%	4.31%	0.00%	1.21%

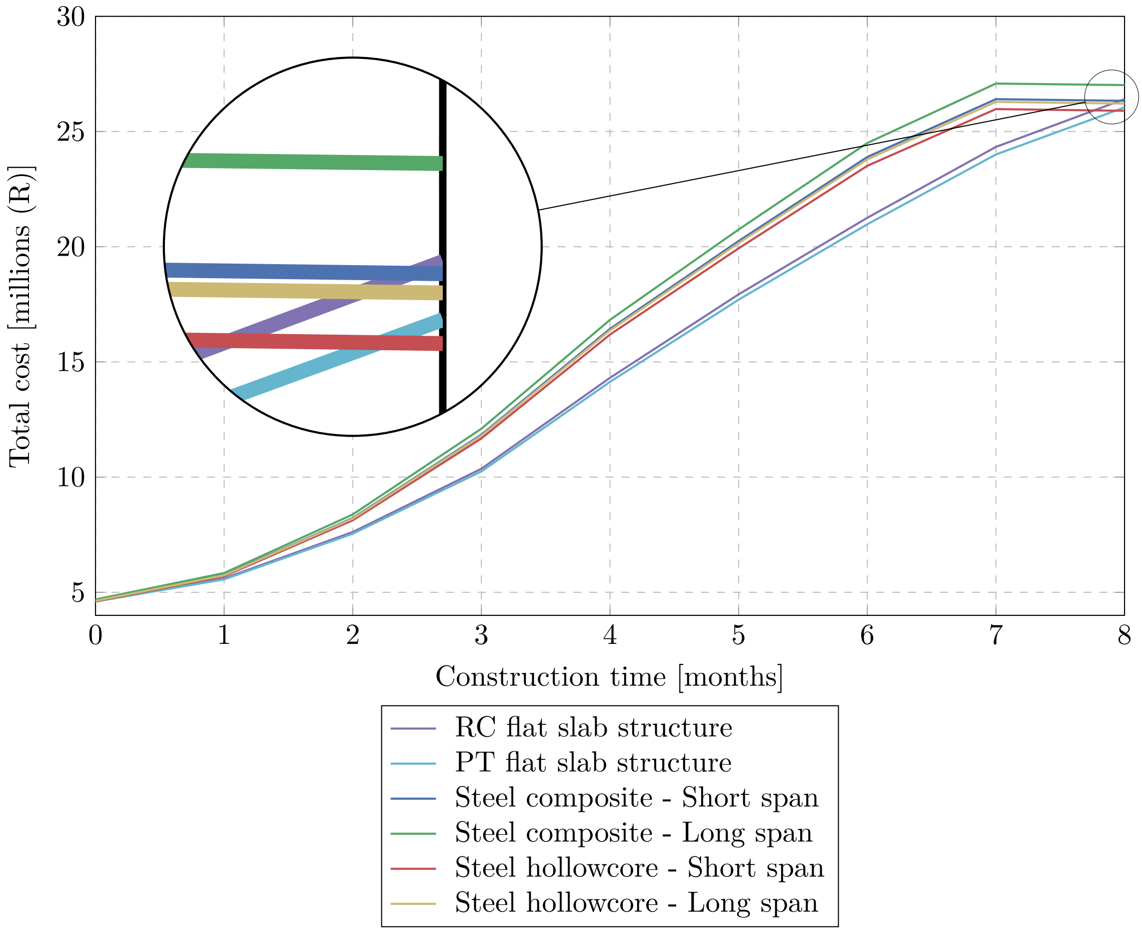


Figure 5.6: Cash flow development for the various steel and concrete structural alternatives

5.5 Discussion of cost-effectiveness of the various structural alternatives

Following the integration of the building costs and the time-related costs, a statement regarding the overall cost-effectiveness of each of the structural alternatives can now be made. Some of the main conclusions that could be drawn from the cost comparison in this chapter were the following:

- **Steel most cost-effective solution** - The short span hollowcore structure provides the most cost-effective solution once all aspects were considered. Table 5.11 shows that the short span hollowcore structure is 0.57 % cheaper than the post-tensioned concrete structure, which equates to a cost difference of approximately R 150 000.00. The most expensive structural alternative is the long span steel composite structure.

- **Time-related costs are an important consideration** - When analysing the final cost it is clear that time related costs significantly influenced the cost comparison between the steel and concrete structural alternatives. The post-tensioned concrete was able to offer the lowest building frame cost by at least R 500 000.00 compared to the steel solutions. However, once the time related costs were considered the short span steel hollowcore structure was found to offer the most cost-effective solution. This illustrates the importance of considering time-related costs such as P&G costs, interest and the ability to earn income at an earlier stage.

- **Important to consider all cost aspects** - Frame costs only make up approximately 10 % of the total construction cost and an even lower percentage of the total capital investment. It could potentially be more cost-effective to pay more for the frame of the structure, if it allows for other advantages to be realised.

5.5.1 Qualitative cost considerations and benefits

Several qualitative cost factors have been identified during the study that are difficult to quantify in physical cost terms. It is however recognised that these factors exist and could influence the comparison of different structural alternatives. Some of these factors include the following:

- **Long spans and the provision of column free floor space** - Long span structural solutions provide increased column free floor space, which enables greater flexibility in the building's intended use, and potential for future changes in use. This is beneficial from a building owner's point of view, who may be constructing the structure on a speculative basis.

- **Speed of construction and reduced disruption** - For many inner city projects, the reduced disturbance to nearby structures and roads is an important consideration. This is particularly important for building extensions and renovations where normal operation of the structure needs to continue (Davison, 2012).

- **Environmental implications** - For modern commercial multi-storey structures the environmental impact of the structure is an important consideration. Steel components are 100 % recyclable without any degradation (Davison, 2012).

Chapter 6

Sensitivity analysis

A sensitivity analysis is used to gain a better understanding of how project performance is influenced by adjusting the assumptions that the performance is based upon. This is often referred to as a "what-if" analysis and is typically used to compare different scenarios and their potential outcomes based on changing conditions (Laidre, 2016). The aim of the sensitivity analysis performed in this study was to explore the change in the cost-effectiveness, of the project with different structural solutions, when certain components of the cost model are adjusted. This is of value because it reveals how sensitive the cost model, and the competitiveness of any particular material, are to a change in certain parameters. Furthermore, it provides the opportunity to investigate the project performance under different assumptions to allow a better understanding of how the project would perform in these situations. Overall, the sensitivity allows for more informed decisions regarding how to proceed with a project.

The sensitivity analysis performed in this study can be broken down into three main sections. Each of these aspects is discussed in more detail in Sections 6.1 to 6.3 in this chapter.

1. Change in construction time
2. Change in cost of steel
3. Changing income and total building cost

6.1 Change in construction time

The first component of the sensitivity analysis investigates the influence that a change in the duration of the construction programme can have on the total cost of the project. This was achieved by varying the length of the construction programme, while keeping the construction and capital costs constant. The influence of the time-related costs on the total project cost could thus be highlighted. Time-related costs that were considered include preliminary and general (P&G) costs, interest, and the ability to earn income at an earlier stage. The P&G costs have been adjusted proportionally to the length of the construction programme. During a discussion with a director from Isipani Construction it was stated that the overwhelming majority of P&G costs are time-related. It was therefore decided that varying the P&G costs directly in proportion to the length of the construction period is a reasonable assumption. The same effective monthly interest rate and monthly income were used as in the initial cost comparison presented in Section 5.3.

Cost information relating to the short span steel structure with hollowcore floors was used for the purposes of this comparison. The cost information for any of the structural alternatives could have been used, and therefore conclusions regarding the sensitivity to a change in construction time will be applicable to each of the structural alternatives. Chapter 4 revealed the construction programme for the steel framed structure with hollowcore units to be approximately seven months long. Therefore, it was decided to investigate programme durations ranging from five to nine months. All costs were compared to the seven month construction programme in order to provide insight into the cost implications that can be associated with a change in the duration of the construction programme. Table 6.1 and Figure 6.1 show the change in total project cost as the length of the construction programme is varied.

Table 6.1: Comparison of total capital investment for the same structure with different length construction programmes

	DURATION OF CONSTRUCTION PROGRAMME [MONTHS]				
	5	6	7	8	9
Total capital investment	R 25 058 539	R 25 454 340	R 25 824 260	R 26 200 859	R 26 602 417
Cost difference	-R 765 720.	-R 369 920	R 0	R 376 599	R 778 156
Percentage difference	-2.97%	-1.43%	0.00%	1.46%	3.01%

Table 6.1 and Figure 6.1 reveal that a change in the duration of the construction programme has significant cost implications. A change of one month in construction time results in a cost difference of approximately R 370 000.00, which equates to 1.45 % of the total project cost. The cost difference increases as the change in the duration of the construction programme increases from one to two months. A two month reduction in construction time results in time related savings of approximately R 765 000.00 which equates to approximately 3 % of the total project cost.

It should be noted that the influence of the time-related costs becomes somewhat greater when the change in construction time increases. The steel framed structure supporting hollowcore floor units required a construction time of approximately seven months, and it is therefore unlikely that the length of this construction programme could be changed by more than a month or two. However, for a larger structure, which would require a longer construction time, there could be potential for a greater change in the duration of the construction period. Greater time reductions would present an opportunity for even more significant cost savings.

For the structure considered in this section, reducing the construction programme by one month was equivalent to a cost reduction of 1.5 % of the total construction cost. For example, if a

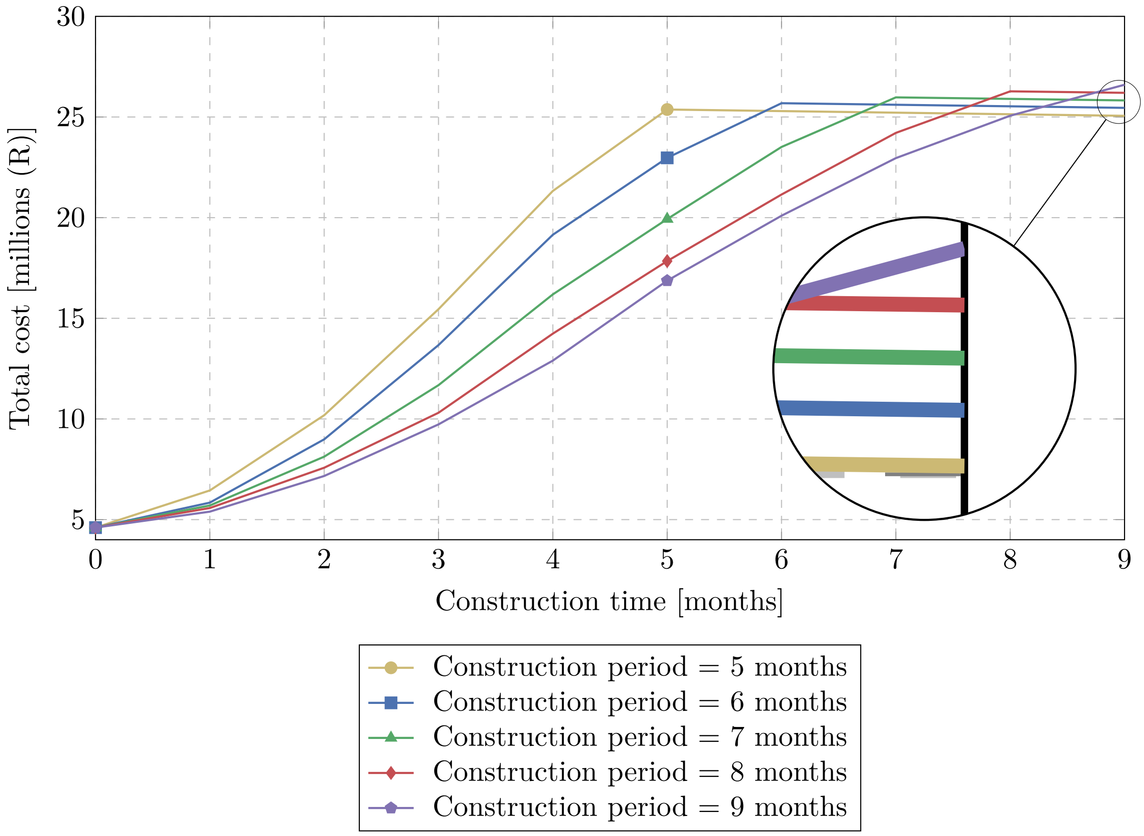


Figure 6.1: Cash flow comparison for a structure with the same cost, excluding P&G costs, but construction programmes of different duration

construction programme of 14 months was able to be reduced to 12 months, the cost savings may still only be equivalent to 1.45 % of the total construction cost, but due to the higher cost of the project this would result in greater cost savings.

6.2 Change in the total steel cost

The costs used in this study reflect current construction costs for multi-storey office buildings in South Africa. These costs are, however, subject to fluctuations depending on a number of factors, such as the building location and complexity, market conditions and many other factors. It is therefore deemed of value to know how the cost comparison would be influenced if the cost rates employed in this study were adjusted. This section investigates the influence that a change in the cost of steel would have on the frame cost of the various structural solutions considered in this study.

The following reasons have been identified for choosing to vary the cost of steel in this study:

1. The cost of steel that is quoted during a project is a compound cost which is made up of several different components. During a discussion with the CEO of Union Steel these components were discussed in order to gain an understanding of how steel rates are developed during a project. The results of this discussion are included in Table C.2 in Appendix C.

Table C.2 reveals that a number of components, in addition to the material cost, make up the total cost rate of steel with aspects such as fabrication, erection and any coatings that are required all forming significant contributions. An optimization of the current steel design, fabrication, delivery and erection processes could lead to a reduction in the cost of steel in a project.

2. At present, steel framed multi-storey office buildings are rarely constructed in South Africa. This results in the cost of structural steel being relatively high compared to other parts of the world, where steel is more frequently used and better understood. An increased investment in steel framed office buildings would lead to more competition when tendering for projects and could assist in reducing the cost of steelwork. Additionally, due to the fact that steel framed multi-storey structures are not frequently used, there could be significant potential for improvement and optimisation. Concrete structures, on the other hand, are frequently used for multi-storey office buildings in South Africa, and it is therefore unlikely that there is much potential for reducing the construction cost of such buildings.

3. Multi-storey office buildings lend themselves to a high degree of repetition and standardisation. This provides the potential for a reduction in fabrication costs compared to other types of steel structures. There could therefore be an opportunity for lower steel prices to be achieved than what are currently being quoted.

4. The final reason that was identified is that the frame cost of steel framed structures is very dependent on the cost of steelwork. Concrete framed structures, on the other hand, have other factors, such as the cost of formwork and reinforcement, that constitute a significant proportion of the total frame cost. Therefore, adjusting the cost of one element, such as cement or reinforcing, would consequently not make as significant a difference as adjusting the cost of steel.

Due to the reasons given above it was decided to vary the cost of steel and investigate the influence that this would have on the comparison between the frame costs for each of the structural alternatives considered in the study. This was achieved by varying the current steelwork rate by a factors ranging from 0.7 to 1.3. The cost factors were applied to the following parts of the structure:

- Steel beams and columns
- Connections
- Shear studs
- Bond-Dek sheeting

Figure 6.2 presents the frame costs of the various steel and concrete structural alternatives when the cost of the steel components listed above are adjusted. The frame cost for the concrete structural alternatives do not change because the only steel present in these structures is the steel reinforcement and this cost was not adjusted.

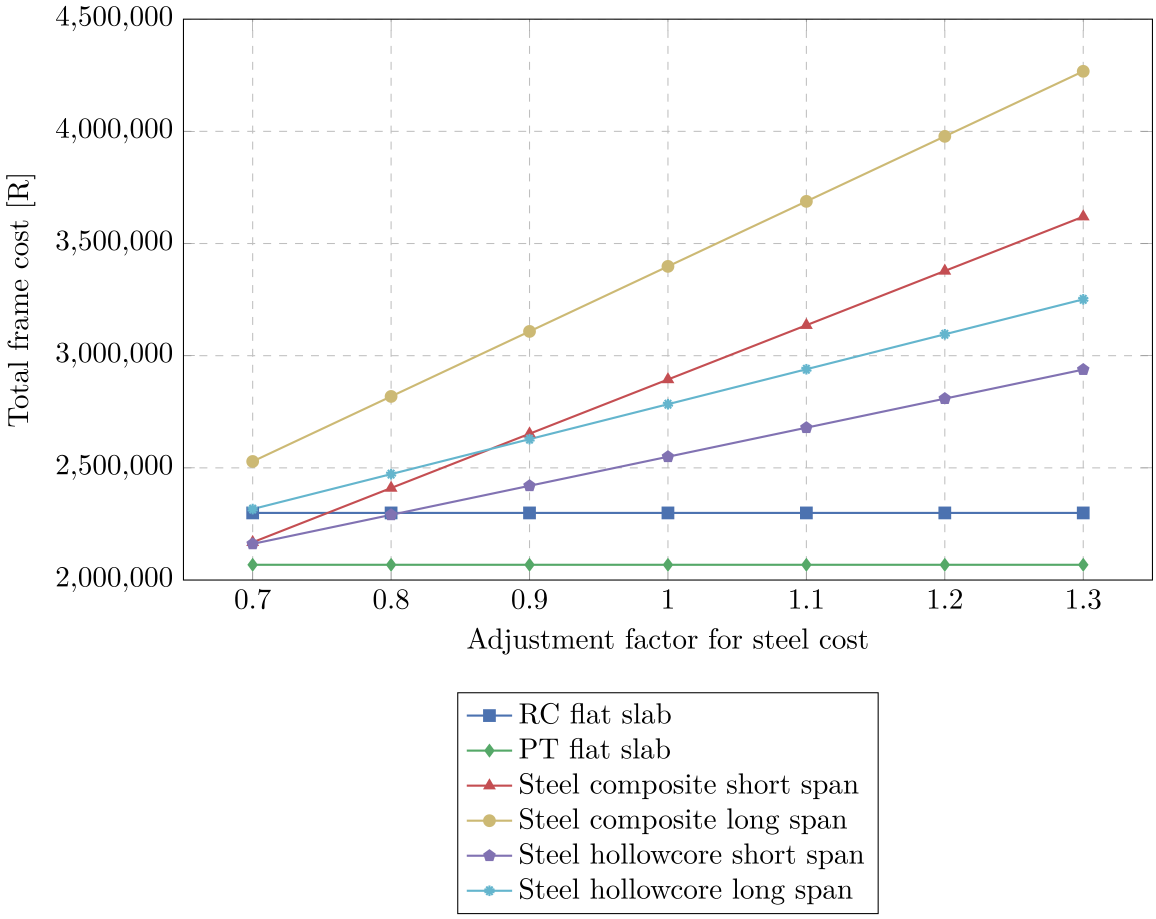


Figure 6.2: Frame costs for various steel and concrete structural alternatives as the cost of steel is changed

The sensitivity of the frame cost of a structural alternative to a change in the cost of steel, is equivalent to the gradient of the corresponding line on the graph in Figure 6.2. A steeper gradient indicates a greater sensitivity to a change in the cost of steel. Some of the main conclusions that could be drawn from Figure 6.2 were the following:

- Considering the gradient of the various lines in Figure 6.2, it is clear that the steel composite structures possess the greatest sensitivity to a change in the cost of steel. This was to be expected with the steel composite structures making use of a large amount of steelwork, Bond-Dek steel sheeting and a large number of shear connectors. A reduction of 10 % in the cost of steel equates to a cost difference of approximately R 240 000.00 or R 290 000.00 for the short and long span composite structure respectively. It can therefore be stated that the composite steel structures would stand to benefit the most from a reduction in the cost of steel. This, of course, means that the frame cost of composite steel structures would also experience the largest increase in cost if the cost of steel were to increase.

- The steel structures supporting hollowcore units also stand to benefit from a reduction in the cost of steel, although the benefit is not as significant as for the composite steel structures. A 10 % reduction in the cost of steel equates to a cost difference of approximately R 130 000.00 or R 155 000.00 for the short and long span steel hollowcore structures respectively. However, the lower sensitivity to the cost of steel does mean that the frame cost of the steel hollowcore structures experience a smaller increase if the cost of steel were to increase, compared to the composite structures.

- A 20 % reduction in the cost of steel allows the steel structural alternatives to be quite competitive with the RC flat slab structure, and only slightly more expensive than the PT flat slab structure. It is important to note that these costs only include the cost of the frame, and no time-related costs have been taken into account. The influence of the time-related costs have already been shown to have significant cost implications in Section 6.1. Therefore, considering the time-related costs would further benefit the steel framed structures.

In conclusion, it was shown that the steel composite structures would stand to benefit the most from a reduction in the cost of steel. Therefore, a significant reduction in current steel costs ($>$ 30 % reduction) would most likely see the steel composite structures become the most cost-effective structural alternative.

6.3 Influence of building income and cost of non-structural components

The final component of the sensitivity analysis involved varying the cost of the non-structural components and the building income, and investigating the influence this would have on the cost-effectiveness of the various structural solutions.

The costs employed in this study up to this point were chosen to be representative of current costs that are applicable to a typical low-rise office building in South Africa. However, different office structures may have requirements differing from those for the typical office building considered in this study. An example of this would be a prestigious office building, where several costs would need to be increased compared to a typical office building. These costs could include aspects such as higher quality finishes and fitments, additional mechanical and electrical services, more wetpoints, specialist cladding and more expensive lifts, to name a few examples. It is unlikely that there would be a significant increase in the cost of the building's frame, as the occupation of the building remains unchanged, and as such the loading would remain similar. The major cost variations would be in the form of the non-structural components mentioned above. However, the income generated from the building would need to be increased to ensure that the development remained profitable. Alternatively, if very basic offices were required, then it would be necessary to reduce the cost of the non-structural components, and the income would then also be reduced.

Considering the aspects discussed above, it was decided to make the following changes to the cost model to develop this component of the sensitivity analysis:

- **Non-structural component cost** - The cost of the non-structural components considered in this study were increased or decreased by 50 % for the prestigious and basic office building respectively.

- **Land cost** - Similarly, the cost of land has been increased or decreased by 50 %. The reason for this is that it is unlikely that a prestigious office building will be located in a poor location, and as a result the land costs would typically be higher. On the other hand, it is unlikely that the basic office building will be constructed in an area with high land costs.

- **Building income** - The income was increased to an amount such that it allowed the project to remain profitable. De-Leeuw Quantity Surveyors advised that a return of 12 % on the total capital investment during the first year of operation is reasonable for an office building. Therefore the income was either increased or decreased to an amount that allows this rate of return to be achieved. The monthly income ranges from approximately R 150 000.00 for the basic office building to R 360 000.00 for the prestigious office building.

The results of the sensitivity analysis following the implementation of the changes discussed above, are shown in Table 6.2 and Figure 6.3.

Table 6.2: Total capital investment for office buildings with different costs and incomes

	TOTAL CAPITAL INVESTMENT [RAND]					
	RC FLAT SLAB	PT FLAT SLAB	COMPOSITE SHORT SPAN	COMPOSITE LONG SPAN	HOLLOWCORE SHORT SPAN	HOLLOWCORE LONG SPAN
Frame cost as % of total construction cost	20.65%	19.08%	25.35%	28.36%	23.06%	24.60%
Basic office	15 328 261	14 967 930	15 567 427	16 250 757	15 134 421	15 448 405
Cost difference	360 330	0	599 497	1 282 827	166 491	480 475
Frame cost as % of total construction cost	11.97%	10.94%	14.99%	17.10%	13.45%	14.49%
Typical office	26 407 700	26 047 381	26 332 703	27 016 032	25 899 696	26 213 681
Cost difference	508 003	147 684	433 006	1 116 336	0	R 313 984
Frame cost as % of total construction cost	8.43%	7.66%	10.64%	12.24%	9.50%	10.27%
Prestigious office	37 487 139	37 126 820	37 158 103	37 841 432	36 725 096	37 039 081
Cost difference	762 042	401 724	433 006	1 116 336	0	313 984

From the results shown in Table 6.2 and Figure 6.3 the following conclusions can be drawn:

- The post-tensioned concrete structure provides a marginally cheaper solution for the basic office building compared to the steel hollowcore structure. This can be attributed to the fact that of the two options the post-tensioned structure offers a lower frame by

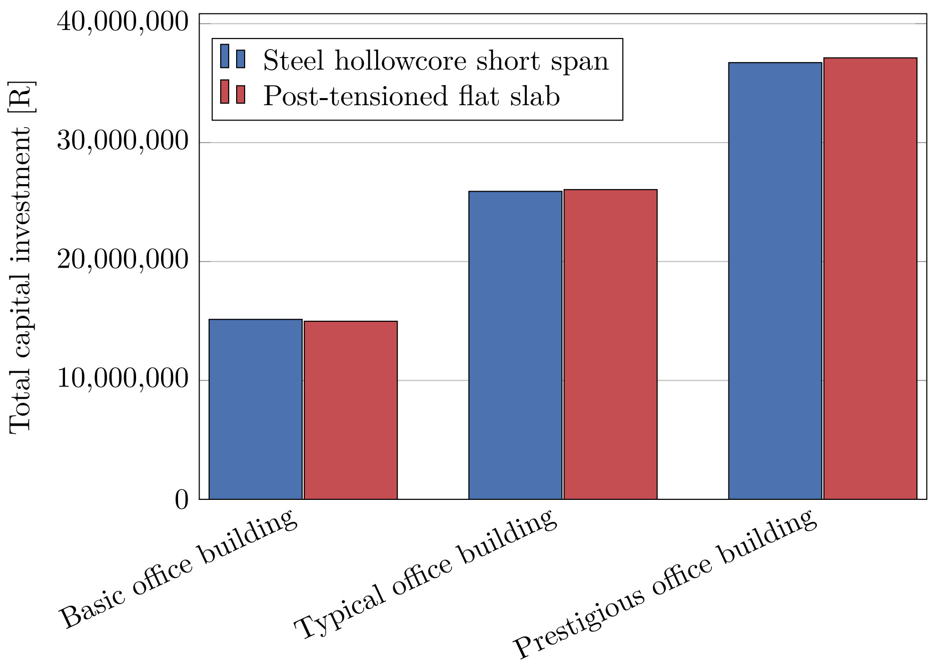


Figure 6.3: Comparison of a steel and concrete structural alternative for different building costs and incomes

approximately R 500 000.00. Due to the fact that the cost of non-structural components and the building income are now relatively low, the influence of the time-related costs become less significant, while the importance of the frame cost becomes more important. Therefore, even though the steel structure has a one month shorter construction period than the post-tensioned structure, the influence of the time-related costs are not sufficient to overcome the difference in frame cost. Therefore the post-tensioned structure provides the most cost-effective option.

- The steel hollowcore short span structure provides the lowest total cost for both the typical and prestigious office buildings. For the typical office structure the short span steel hollowcore option was found to be approximately R 150 000.00 cheaper than the post-tensioned flat slab option. Whilst for the prestigious office building, the cost difference between these two options increases to approximately R 400 000.00. The is because as the cost of the project increases, so does the influence of the time-related costs, while the cost of the building's frame becomes less important. It can therefore be concluded that as the cost of non-structural components and income for a structure increase, the influence of the frame cost on total cost becomes less significant.

- It can also be stated that the cost model is not very sensitive to a change in the cost of the non-structural components and income. A change of 50 % in the cost of the non-structural components is a significant change in the total capital investment for the project. The cost differences between the structural alternatives on the other hand do not experience a very significant change. It can therefore be concluded that the model is not very sensitive to a change in the cost of non-structural components, and is more sensitive to other factors such as the duration of the construction programme which was discussed in Section 6.1.

Chapter 7

Conclusions and recommendations

7.1 Summary of findings

A comparison of the costs of various steel and concrete structural alternatives led to several important findings, with some of the main ones being:

- A layout and building configuration was developed that could be seen as being representative of a typical low-rise multi-storey office structure in South Africa. The layout consisted of four floors, with a gross internal floor area of 1950 m^2.
- For the layout that was developed, two steel and two concrete structural alternatives were identified as solutions that could be suitable for a project of this nature.
- During the design of the structural alternatives several key aspects were taken into account, with important findings regarding these aspects including the following:
  - Composite construction was shown to be a critical consideration for the steel structural alternatives, and was shown to offer significant advantages over non-composite construction methods.
  - The fire design of multi-storey steel and concrete structures was considered, with particular attention paid to the fire design of the steel framed structures considered in the study. The Slab Panel Method was shown to be suitable for use in the steel composite structure.
  - The level of floor vibrations that could be expected was considered, and although no changes to normal design procedures were required, the calculations revealed the importance of considering floor vibrations.
- Comparing the construction programmes revealed that the steel structural alternatives offered a reduction of approximately one month in the overall construction time, compared to the concrete structures.
- Some of the main findings from the cost comparison were the following:

- Steel structural alternatives experienced lower foundation costs compared to the concrete structures, with foundation costs being reduced by up to 37 %.
- The concrete framed structures had lower frame costs than the steel structural alternatives, with the post-tensioned flat slab structure providing the lowest total frame cost.
- Vermiculite spray was shown to provide the cheapest method of fire protection for both a 60 and 120 minute fire rating. Intumescent paint was found to be approximately 50 % more expensive than vermiculite spray for a 60 minute fire rating, and between 2.5 to 3 times more expensive for a 120 minute fire rating.
- The implementation of the slab panel method for the composite metal deck floors during the fire limit state enabled a reduction of approximately R 150 000.00 in the total cost of fire protection.
- Time-related costs were shown to significantly influence the cost comparison between the steel and concrete structures.

- The sensitivity analysis allowed for a number of findings, including:
  - There were substantial cost implications associated with a change in the duration of the construction programme, with the influence increasing as the change in construction time increased.
  - The steel composite structure was found to have the greatest sensitivity to a change in the cost of fabricated and erected steelwork.
  - As the total project cost increases, the influence of the frame cost becomes less significant, with time-related costs proving to be more critical.

7.2 Conclusion

The study revealed that for a typical low rise office building constructed in South Africa, a steel framed structure compositely connected to hollowcore floor slabs can provide the most cost-effective solution. The cost difference between the steel composite and post-tensioned structure was calculated to be approximately R 150 000.00 which equated to 0.6 % of the total project cost. This reveals that there is certainly potential for the increased use of steel framing systems in low-rise office buildings in South Africa, especially if more expertise is gained while designing and constructing more such buildings.

7.3 Contributions

This study makes a contribution to the state of knowledge related to the design and construction of steel and concrete framed low-rise office buildings in two main areas:

1. The study provides comparative information between a steel and concrete framed multi-storey office structure, of which there is currently very little information available for the South African context. This comparative information is able to assist in reducing uncertainty as to how the construction cost and time of steel and concrete framing systems compare with one another when used for the structures of low-rise office buildings.

2. The study reveals a methodology for comparison between different structural solutions, and identifies key aspects that should be taken into account when developing cost comparisons between different structural alternatives. The study was able to highlight the importance of considering time-related costs when developing cost comparisons between steel and concrete structural alternatives.

7.4 Recommendations for further research

The study lends itself to future research in several areas, with some of the main areas that were identified including:

- *Building use* - The structure considered in this study was designed for office use and the design loads, vibration criteria and fire rating were all chosen with this occupation in mind. The study could be expanded to investigate buildings with other occupations such as residential, hospital or education use and investigate how this would influence the cost model.

- *Structural configuration* - The structure considered in this study was identified to be a typical low-rise office building, and the layouts, spans and building height were all chosen to represent a structure of this nature. This configuration is of course subject to change depending on the nature of the project, among other factors, and additional research could investigate the influence of a different floor layout, spans, and number of stories would have on the cost comparison.

- *Structural alternatives* - Two steel structural alternatives and two concrete structural alternatives were considered in this study. Further research could explore additional structural alternatives that were not considered during this study, such as hybrid concrete construction, Slimdek composite floors, and rib and block floors, to name a few examples.

- *Whole building study* - The focus of this study was the comparison of the structural frame but the study could be extended to include the non-structural components and how this would influence the cost comparison.

- *Optimization of design and delivery methods for steel framed office structures* - Section 2.3.3 discussed that due to the nature of many multi-storey office buildings lower steel prices can be achieved compared to other steel structures. This reveals the potential for research into the optimization of the steel delivery methods for multi-storey office buildings, and whether or not there is potential for reducing current costs of steelwork for these structures.

A pre-engineered approach could be adopted for these typical low-rise office structures in order to optimize the steel design, fabrication, delivery and erection process.

- *Environmental benefits* - The environmental considerations were mentioned briefly in Section 5.5.1, but no comparison was done in this study. Further research could consider the environmental impact of steel and concrete structures and how they compare with one another.
- *Life-cycle costs* - The cost comparison developed in this study did not consider the life cycle costs that can be associated with the various steel and concrete structural alternatives. Additional research could investigate the influence that the inclusion of these costs would on the cost comparison between the different building types.
- *Ease of demolition* - The ease of demolition of the different structural alternatives under South African conditions was not considered in this thesis. There are different factors that need to be considered during the demolition of steel and concrete building types. The comparison in this study could be extended to take these demolition considerations into account and explore how the comparison would be influenced.

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Appendix A

Design calculations

This appendix provides additional information regarding the design calculations that were performed during the course of this study, with information being provided on the design of both the steel and concrete framed structural alternatives.

A.1 Steel framed structure with composite metal deck floors

The results of the design calculations for the composite beams supporting Bondek sheeting is presented in Figure A.1 to Figure A.5 below. Information is provided regarding the design during the construction and composite stage, as well as the calculation of the deflection of the beam.

Design of composite floor with Bond-dek sheeting

Structural details
8m span, 2.5m spacing between centres
25MPa concrete
Simply supported
Unpropped construction
Use 19mm shear studs, 115mm long, 2 per rib
Use 0.8 mm thick Bond-Dek sheeting

Factors	
φ	0.9
φ_c	0.6
φ_{sc}	0.8
φ_r	0.85

Bond-Dek self weight		
0.8mm	0.076518	kN/m^2
1mm	0.0966285	kN/m^2
1.2mm	0.116739	kN/m^2

Length of beam	8	m
Spacing between beam centres	2.5	m
f_{cu}	2.50E+07	Pa
Thickness of cover slab	0.065	m
Steel strength	3.55E+08	Pa
Yield strength of reinforcement, f_{yr}	4.50E+08	Pa
Level of shear connection - 60%	0.6	
Depth of sheeting	0.075	m

Section selection	
305x102x28	
Flexural classification	
flange	class 2
web	class 1
Check flexural classification when bea section is changed	

Properties		
h	308.9	mm
b	101.9	mm
t_w	6	mm
t_f	8.8	mm
A	3640	mm^2
Z_{pl_x}	408000	mm^3
I	5.44E+07	mm^4
r_y	20.8	mm
mass	0.28	kN/m

Figure A.1: Composite beam design - Structural details

Construction stage - Placing of deck		
Loading		
Steel beam and deck	0.19	kN/m^2
Construction live load say	0.5	kN/m^2
Factored design loading	1.0	kN/m^2
Total distributed load on the beam (w)	2.6	kN/m
Maximum moment at midspan	20.5	kNm

Loading		
Self weight of 0.8mm Bond-Dek	0.08	kN/m^2
Self weight of beam	0.1	kN/m^2
Total dead load	0.2	kN/m^2

Section selection						
Use 305x102x28						
Provide temporary lateral restraint at midspan						
to limit slenderness ratio						
L/r_y	192.31	<	Allow ===>	200	OK	
M_{rx}	40.5	kNm	> Mu ===>	20.49	OK	Table 5.5

Construction stage - Placing of concrete						
Loading						
Concrete weight	2.52	kN/m^2				
Additional for ponding	0.378	kN/m^2				
Steel beam and deck	0.2	kN/m^2				
Total dead load	3.1	kN/m^2				
Construction live load say	1	kN/m^2				
Factored design loading	5.302	kN/m^2				
Total distributed load on the beam (w)	13.3	kN/m				
Maximum moment at midspan (M_u)	106.0	kNm				
Deck is positively connected to the top flange						
which provides continuous lateral support						
Mr = φ * Z_{pl}*f_y	130.36	kNm	> Mu ===>	106.04	OK	SANS 13.5

Figure A.2: Composite beam design - Construction stage

Composite beam		
Loading		
Total live load	3.5	kN/m^2
Total dead load	4	kN/m^2
Factored design load pressure	9.90	kN/m^2
Total distributed load on the beam (w)	24.76	kN/m
Maximum moment at midspan, M_u = (w*L^2/8)	198.0	kNm
Total load on beam	198.0	kN
End shear force (V_u)	99.0	kN
Resistance		
Effective width = min of:		
0.25*span	2	m
transverse centre of beams	2.5	m
Effective width =====>	2	m
Bending resistance		
Use a partial shear connectuon of 60%		
C'_r = 0.68*φ_c*b*t*f_{cu}	1326.00	kN
T_r = φ*A_s*f_y	1163	kN
Effective thickness of slab		
a	65	mm
Reduced thickness for 60% shear transfer	39	mm
C'_r using reduced thickness = 0.68*φ_c*b*a*f_{cu}	795.60	kN
Cr - Steel compressive force = (φ*A_s*f_y-C'_r)/2	183.7	kN
Area of steel in compression - A_{sc}	0.0005749	m^2
	574.9	mm^2
Area of top flange	896.72	mm^2
Neutral axis lies within the top flange		
Distance to NA from top of flange	5.64	mm
Distance of centroid of area A_{st} from underside of beam	126.01	mm
Dimensions		
e	180.1	mm
e'	303.39	mm
Resistance moment		
Mr = C_r*e + C'_r*e'	274.46	kNm

>Mu ===>	198.0	OK

Loading at composite stage		
Dead load		
Weight of steel beam and deck	0.2	kN/m^2
Weight of concrete	2.52	kN/m^2
Increase to account for ponding (15%)	0.378	kN/m^2
Services	0.4	kN/m^2
Ceiling	0.1	kN/m^2
Total dead load	3.585	kN/m^2
Live load		
Imposed load on floor	2.5	kN/m^2
Movable partitions	1	kN/m^2
Total live load	3.5	kN/m^2

Figure A.3: Composite beam design - Composite stage bending resistance

Vertical end shear						
Steel section has class 1 web						
Shear resistance of flange (V_r) = 0.66*φ*t_w*h*f_y	390.83	kN	> Vu ===>	99.0220976	OK	13.4.1.2
Shear connectors						
Use Bond-Dek deck and stud dimensions						
Studs need to resisit longitudinal shear force of C'$_r$ = Q_r	795.6	kN				
Resistance per stud	58.4	kN		Red Book Table 9.3		
Number of studs required between support and midspan	13.6	studs				
Therefore use ===>	14	studs				
Number of studs required for whole beam	28	studs				
Studs are placed in pairs, therefore spaces required ===>	7	spaces				
Required spacing of studs	571.4285714	mm				
Actual spacing of studs	452	mm	< S_{req} ===>	571.4285714	OK	
Concrete slab						
Longitudinal shear force to be resisted by slab, V_u	755.06	kN				
				SANS 17.5.4.2		
Minimum transverse reinforcement in each half span	260	mm^2		Amount of transverse reinforcement		
A_{rt} > 0.001*t*L/2				Use A_{rt} ===> 500	mm^2	
Longitudinal shear resistance of two shear planes in each half span is the lesser of:						
V_r = 0.8*$φ_r$*A_{rt}*f_{yr}+2.76*$φ_c$*A_{cv}	1167.12	kN				
V_r = 0.4*$φ_c$*f_{cu}*A_{cv}	3120	kN				
Check whether both values of V_r exceed V_u =======>	755.06	kN			OK	

Figure A.4: Composite beam design - additional considerations

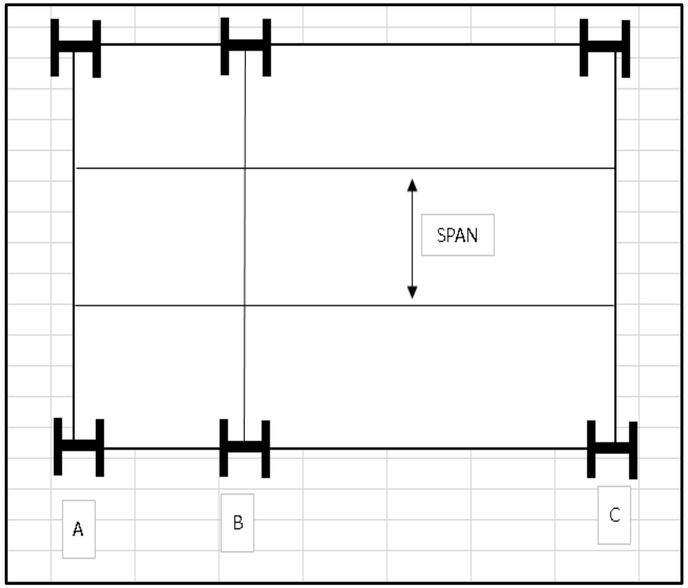
Deflections					
a) non-composite beam under self weight					
Assume beam is unpropped during construction				SLS load factors	
Self weight of beam and metal deck	3.0851748	kN/m^2		dead load	1.1
Construction live load - say	0	kN/m^2		imposed load	1
Total factored distributed load at SLS (w_sls)	8.4842307	kN/m			
$Δ_1$ = 5/384*w*L^4/(E*I)	41.59	mm			
b) Composite beam - Short term live load					
Total live load for composite beam =	3.5	kN/m^2			
Therefore short term live load = 0.5*total live load	1.75	kN/m^2			
Total distributed load at SLS (w_sls)	4.375	kN/m			
Modular ratio = E_{steel}/$E_{concrete}$	7.692				
A_{conc}	16900	mm^2			
Height of NA from bottom of beam	369.98	mm			
I_t = transformed moment of inertia composite beam	2.66E+08	mm^4		Calculate value or obtain from table 9.4 in red book	
I_e = I_{steel} + 0.85*alpha^0.25*($I_{transformed}$-I_{steel})	2.13E+08	mm^4			
$Δ_2$ = 5/384*w*L^4/(E*I)	5.5	mm			
c) Composite beam - long term live load					
$Δ_3$ = 1.15* (5/384*w*L^4/(E*I))	6.3	mm			
d) Shrinkage of concrete					
A_c	0.13	m^2			
y - height of centroid of effective concrete above NA	46.42	mm			
delta shrinkage = ε*A_c*L^2*y/(8*n*I_t)	8.26	mm		Free shrinkage strain of concrete	3.50E-04
Reduce by 30%	5.78	mm			
Total deflection = $Δ_1$+$Δ_2$+$Δ_3$+$Δ_4$	59.17	mm		Deflection is excessive	
Allowable defelction = L/300	26.67	mm			

Figure A.5: Composite beam design - Final deflection

A.1.1 Floor vibration calculations for steel composite structure

The vibrations of the steel composite floor system was checked to ensure that the level of floor vibrations that could be expected was of an acceptable level from a human comfort point of view. Figure A.6 to Figure A.9 reveal the calculations that were performed when checking the level of floor vibrations that could be experienced in both the short and long span steel structures with composite Bond-Dek floors.

Design of composite floors for vibrations from human induced loading



Structural details		
Long span	8	m
Short span	5	m
Spacing of secondary beams	2.5	m
Depth of metal deck	0.075	m
Depth of concrete slab	0.065	m
Concrete strength	25	MPa
$E_{concrete}$	2.60E+04	MPa
E_{steel}	2.00E+05	MPa
$n = E_{steel}/(1.35 \cdot E_{conc})$	5.70	

Loading

Dead		
Concrete and deck	2.53	kPa
15% ponding allowance	0.38	kPa
Celing and services	0.5	kPa
Total dead load ===>	3.41	kPa
SLS dead load factor	1.1	

Imposed		
Recommended dynamic load	0.5	kPa
SLS imposed load factor	1	

	Designation	h	b	t_w	t_f	r_1	m	A	I_x	Z_x	Z_{plx}	r_x	r_y	I_y
	mmxmmxkg/m	mm	mm	mm	mm	mm	kg/m	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3	mm	mm	10^6 mm^4
5m	305x102x28	308.9	101.9	6.1	8.8	7.6	28.2	3.64	54.4	352	408	122	20.8	1.58
8m	305x102x28	308.9	101.9	6.1	8.8	7.6	28.2	3.64	54.4	352	408	122	20.8	1.58
Girder A	305x102x28	308.9	101.9	6.1	8.8	7.6	28.2	3.64	54.4	352	408	122	20.8	1.58
Girder B	406x140x46	402.3	142.4	6.9	11.2	10.2	46	5.9	157	779	889	163	30.3	5.4
Girder C	406x140x39	397.3	141.8	6.3	8.6	10.2	39	4.92	124	625	718	159	28.9	4.1

Figure A.6: Floor vibration calculations for short span composite structure - Page 1

Beam mode

Effective width of concrete slab = 0.4*Lb	3.2	m
Spacing between secondary beams =	2.5	m
Therefore use b ===>	2.5	m
Determine position of neutral axis, y ===>	-2.85	mm

$$I_b = (I_s + A_s * d_s^2) + (I_{conc} + A_{conc} * d_{conc}^2)$$

I_s	5.44E+07	mm^4
A_s	3640	mm^2
d_s	232.3	mm
I_{conc}	1.00E+07	mm^4
A_{conc}	28518.75	mm^2
d_{conc}	29.6	mm

$\}$ I_b = 2.85939E+08 mm^4

UDL acting on beam + own weight	10.90	kN/m

$$\Delta_{ss} = \frac{5}{384} * \frac{w * l^4}{E * I} \qquad f_n = \sqrt{\frac{g}{\Delta}}$$

Δ_beam	7.33	mm
Natural frequency of simply supported beam	6.58	Hz
Average slab concrete thickness, d_e	102.5	mm
Thus per unit width of the slab, D_s	15749.5	mm^4/mm
Transformed moment of inertia per unit width in the beam direction, D_b	114375.5	mm^4/mm
Effective beam panel width		

$$B_b = C_b * \left(\frac{D_s}{D_b}\right)^{0.25} * L_b < \frac{2}{3} * floor\ width$$

Cb = 2.0 except for beams next to a slab edge without a wall on , in which case Cb = 1

C_b	2	
B_b	9.75	m

Bay CAN be considered to be an interior panel because beams continue to extend in both directions on either side of panel, therefore actual floor width is at least = 3*7.5 = 22.5m. Thus the beam panel width of 9.75m is acceptable

Continuity factor , c	1.5	
Weight of beam panel, $W_b = c*(w_b/s)*B_b*L_b$	510.1	kN

Girder mode

Consider interior girder along gridline B

Effective width of concrete slab = 0.4*Lg	3	m
Spacing of girders	5	m
Therefore use b ===>	3	m
Average slab concrete thickness, d_e	102.5	mm
Position of NA from half height of flute above flange	-22.68	mm

$$I_b = (I_s + A_s * d_s^2) + (I_{conc} + A_{conc} * d_{conc}^2)$$

I_s	1.57E+08	mm^4
A_s	5900	mm^2
d_s	261.3	mm
I_{conc}	4.72E+07	mm^4
A_{conc}	53966.25	mm^2
d_{conc}	28.6	mm

$\}$ I_g = 6.51229E+08 mm^4

UDL acting on girder + own weight	28.80	kN/m

$$\Delta_{ss} = \frac{5}{384} * \frac{w * l^4}{E * I}$$

Δ_girder	9.11	mm
Natural frequency of simply supported girder	5.91	Hz
Transformed moment of inertia per unit width in the beam direction, $D_g = I_g/L_g$	86830.6	mm^4/mm

$$B_g = 1.8 * \left(\frac{D_b}{D_g}\right)^{0.25} * L_g < \frac{2}{3} * floor\ length$$

Total width of floor is 13m for girder direction. Therefore need to limit the width of the girder panel.

B_g	14.46	m
Floor length	13.00	m
2/3 * Floor length	8.67	m
Therefore use Bg ===>	8.67	m
Continuity factor , c	1	
Weight of girder panel, $W_g = c*(w_g/L_b)*B_g*L_g$	234.0	kN

Value is not increased by 50% because the girders frame directly into the columns

Combined mode

Need to reduce Δ_girder because Lg < Bb

$$\Delta_g' = \frac{L_g}{B_b} * \Delta_g$$

Δ_girder'	7.01	mm

Frequency of floor: $f_n = 0.18 * \sqrt{\frac{g}{\Delta_b + \Delta_g'}}$

f_n	4.71	Hz

Equivalent combined mode panel weight:

$$W = \frac{\Delta_b}{\Delta_b + \Delta_g'} * W_b + \frac{\Delta_g'}{\Delta_b + \Delta_g'} * W_g$$

Δ_beam	7.33	
Δ_girder'	7.01	
Wb	510.1	
Wg	234.0	
W	375.2	kN
For office floor with partial height partitions, β=	0.03	
P_0	0.29	kN

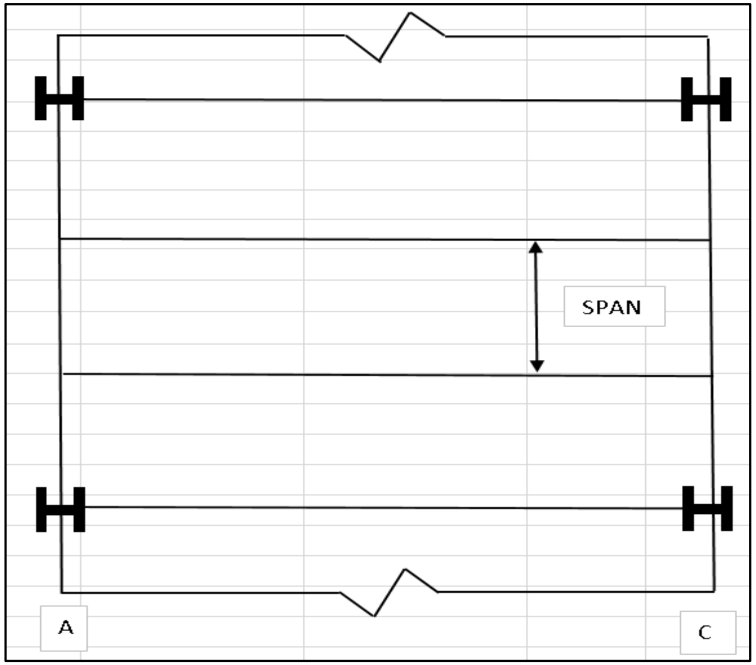
$$\frac{peak\ acceleration}{g} = \frac{a_p}{g} = P_0 * \frac{e^{0.35 * f_n}}{\beta * W} < \frac{a_0}{g}$$

Limit on a_0/g	0.50%
Peak acceleration / g	0.4960%

VIBRATION IS SATISFACTORY, OK

Figure A.7: Floor vibration calculations for short span composite structure - Page 2

Floor vibrations from human induced loading in long span composite structure



Structural details

Beam span	13	m
Spacing of secondary beams	2.5	m
Depth of metal deck	0.075	m
Depth of concrete slab	0.065	m
Concrete strength	25	MPa
$E_{concrete}$	2.60E+04	MPa
E_{steel}	2.00E+05	MPa
$n = E_{steel}/(1.35*E_{conc})$	5.70	

Loading

Dead		
Concrete and deck	2.53	kPa
15% ponding allowance	0.38	kPa
Celing and services	0.5	kPa
Total dead load ===>	3.41	kPa

Imposed		
Recommended dynamic load	0.5	kPa

SLS imposed load factor	1

SLS dead load factor	1.1

	Designation	h	b	t_w	t_f	r_1	m	A	I_x	Z_x	Z_{plx}	r_x	r_y	I_y
	mmxmmxkg/m	mm	mm	mm	mm	mm	kg/m	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3	mm	mm	10^6 mm^4
13m	406x178x54	402.6	177.6	7.6	10.9	10.2	54.1	6.86	187	927	1050	165	38.6	10.2
Girder A	406x178x54	402.6	177.6	7.6	10.9	10.2	54.1	6.86	187	927	1050	165	38.6	10.2
Girder C	406x178x54	402.6	177.6	7.6	10.9	10.2	54.1	6.86	187	927	1050	165	38.6	10.2

Figure A.8: Floor vibration calculations for long span composite structure - Page 1

Beam mode

Effective width of concrete slab = 0.4*Lb	5.2	m
Spacing between secondary beams =	2.5	m
Therefore use b ===>	2.5	m
Determine position of neutral axis, y ===>	27.38	mm

$$I_b = (I_s + A_s * d_s^2) + (I_{conc} + A_{conc} * d_{conc}^2)$$

I_s	1.87E+08	mm^4
A_s	6860	mm^2
d_s	248.9	mm
I_{conc}	1.00E+07	mm^4
A_{conc}	28518.75	mm^2
d_{conc}	59.9	mm
I_b	7.24352E+08	mm^4

UDL acting on beam + own weight	11.16	kN/m

$$\Delta_{ss} = \frac{5}{384} * \frac{w * l^4}{E * I} \qquad f_n = \sqrt{\frac{g}{\Delta}}$$

Δ_beam	28.64	mm
Natural frequency of simply supported beam	3.33	Hz
Average slab concrete thickness, d_e	102.5	mm
Thus per unit width of the slab, D_s	15749.5	mm^4/mm
Transformed moment of ineretia per unit width in the beam direction, D_b	289740.7	mm^4/mm
Effective beam panel width		

$$B_b = C_b * \left(\frac{D_s}{D_b}\right)^{0.25} * L_b < \frac{2}{3} * floor\ width$$

Cb = 2.0 except for beams next to a slab edge without a wall on , in which case Cb = 1

C_b	2	
B_b	12.55	m

Bay CAN be considered to be an interior panel because beams continue to extend in both directions on either side of panel, therefore actual floor width is at least = 3*7.5 = 22.5m. Thus the beam panel width of 12.55m is acceptable

Continuity factor , c	1.5	
Weight of beam panel, $W_b = c*(w_b/s)*B_b*L_b$	1092.6	kN

Girder mode

Consider edge girder gon Gridline A / C

Effective width of concrete slab = 0.2*Lg	1.5	m
Spacing between girders	13	m
Therefore use b ===>	1.5	m
Average slab concrete thickness, d_e	102.5	mm
Position of NA from half height of flute above flange	7.54	mm

$$I_b = (I_s + A_s * d_s^2) + (I_{conc} + A_{conc} * d_{conc}^2)$$

I_s	1.87E+08	mm^4
A_s	6860	mm^2
d_s	231.3	mm
I_{conc}	2.36E+07	mm^4
A_{conc}	26983.125	mm^2
d_{conc}	58.8	mm
I_g	6.70766E+08	mm^4

UDL acting on girder + own weight	29.54	kN/m

$$\Delta_{ss} = \frac{5}{384} * \frac{w * l^4}{E * I}$$

Δ_girder	9.07	mm
Natural frequency of simply supported girder	5.92	Hz
Transformed moment of ineretia per unit width in the beam direction, $D_g = I_g/L_g$	89435.5	mm^4/mm
Multiply D_g by 2 for edge girders	178870.94	mm^4/mm

$$B_g = 1.8 * \left(\frac{D_b}{D_g}\right)^{0.25} * L_g < \frac{2}{3} * floor\ length$$

B_g	15.23	m
Floor length	13.00	m
2/3 * Floor length	8.67	m
Therefore use Bg ===>	8.67	m

Total width of floor is 13m for girder direction. Therefore need to limit the width of the girder panel.

Continuity factor , c	1	
Weight of girder panel, $W_g = c*(w_g/L_b)*B_g*L_g$	147.7	kN

Value is not increased by 50% because the girders frame directly into the columns

Combined mode

Need to reduce Δ_girder because Lg < Bb

$$\Delta_g' = \frac{L_g}{B_b} * \Delta_g$$

Δ_girder'	5.42	mm

Frequency of floor: $f_n = 0.18 * \sqrt{\frac{g}{\Delta_b + \Delta_g'}}$

f_n	3.05	Hz

Equivalent combined mode panel weight:

$$W = \frac{\Delta_b}{\Delta_b + \Delta_g'} * W_b + \frac{\Delta_g'}{\Delta_b + \Delta_g'} * W_g$$

Δ_beam	28.64	mm
Δ_girder'	5.42	mm
Wb	1092.6	kN
Wg	147.7	kN
W	942.3	kN
For office floor with partial height partitions, β=	0.03	
P_0	0.29	kN

$$\frac{peak\ acceleration}{g} = \frac{a_p}{g} = P_0 * \frac{e^{0.35*f_n}}{\beta * W} < \frac{a_0}{g}$$

Limit on a_0/g	0.50%
Peak acceleration / g	0.35%

VIBRATION IS SATISFACTORY, OK

Figure A.9: Floor vibration calculations for long span composite structure - Page 2

A.1.2 Results of Slab Panel Method (SPM) software analysis

The Slab Panel Method was used to design the composite steel floors during the fire limit state. An example of the output from the SPM software that was used is shown in Figure A.10 below:

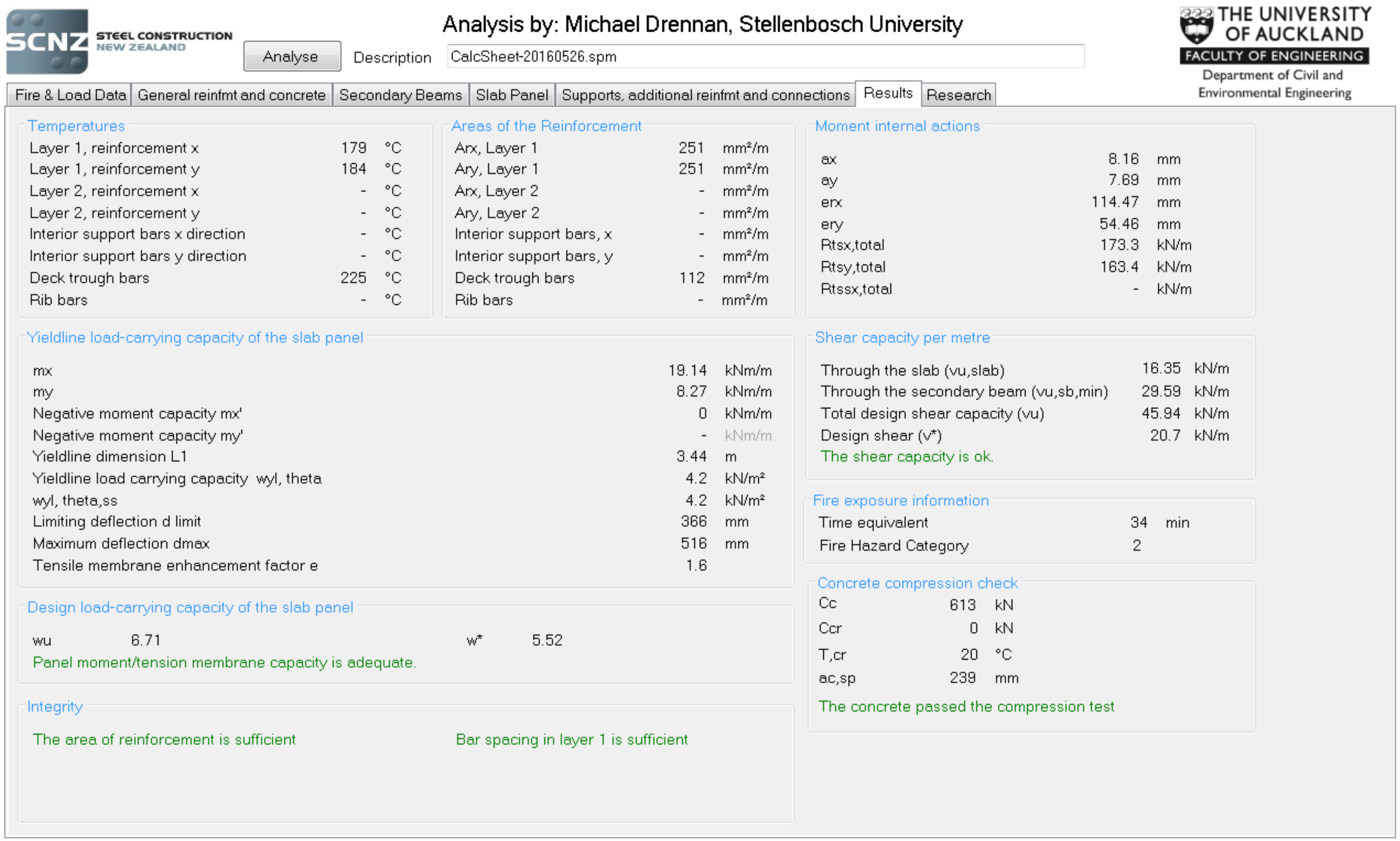


Figure A.10: Results of analysis with Slab Panel Method

A.2 Steel framed structure with precast hollowcore floors

This section reveals that calculation procedure that was followed when designing the steel beams compositely connected to the hollowcore floor slabs. In order to illustrate the design procedure, the design of a typical 8 m internal composite beam is shown in Figure A.11 to A.16.

Design of composite beam and hollowcore slab

Structural details
Hollowcore units span 7.5m
Use data from TopFloor hollowcore publications
Beams designed to act compositely with floor

Specification

Precast hollowcore units		
Unit depth	200	mm
Unit width	1200	mm
Number of cores	11	
Spacing of cores	100	mm
Self weight - Excluding joints and 40mm screed	2.9	kN/m^2
Self weight - Including joints and 40mm screed	4.1	kN/m^2
Length of concrete infill	500	mm

Length of composite beam	8	m
f_{cu}	30	MPa
Density of wet concrete	2400	kg/m^3
Density of dry concrete	2350	kg/m^3
Thickness of structural screed	40	mm
Yield strength of steel , f_y	355	MPa
Span of hollowcore unit	7.5	m
Level of shear connection	Varies	
E modulus of steel	200000	MPa
Yield strength of reinforcement, f_{yr}	450	m

Shear connectors		
Stud diameter	19	mm
Stud height after welding	120	mm

Section selection	Flexural classification	
406x178x60	flange	class 2
	web	class 1

Properties					
h	406.4	mm	r_y		mm
b	177.8	mm	mass	60.1	kg/m
t_w	7.8	mm	J		mm^4
t_f	12.8	mm	C_w		mm^6
A	7610	mm^2	Z_{pl_y}		mm^3
Z_{pl_x}	1200000	mm^3	I_y		mm^4
I_x	2.15E+08	mm^4			

Floor loading

Construction stage		
a) Unbalanced loading		
Floor (hollowcore units only)	2.9	kN/m^2
Steel beam - Changes with each beam	0.08	kN/m^2
Total dead load ===>	3.0	kN/m^2
a) Balanced loading		
Floor (hollowcore units, joints and topping)	4.1	kN/m^2
Steel beam - same as above	0.08	kN/m^2
Additional screed required	0.15	kN/m^2
Total dead load ===>	4.326	kN/m^2
Imposed construction load say ===>	0.5	kN/m^2

Composite stage		
Floor (hollowcore units, joints and topping)	4.10	kN/m^2
Steel beam	0.08	kN/m^2
Services and ceiling	0.50	kN/m^2
Additional load for screed weight	0.15	kN/m^2
Total dead load ===>	4.83	kN/m^2
Offices for general use	2.5	kN/m^2
Partitions	1	kN/m^2
Total imposed load ===>	3.5	

Figure A.11: Structural details and loading information

Construction condition - Steel beam design					
Ultimate limit state loading condition					
a) Unbalanced loading - Units on one side of the beam					
For the purposes of this study the unbalanced load condition will not be considered. It can be assumed that the beams are restrained against torsional buckling or are placed in a sequence that does not induce torsional moments on the beam					
a) Balanced loading - Units on both sides of the beam					
Design load, w	44.9	kN/m			
Design shear, $F_v = wL/2$	179.7	kN			
Design moment, $M_x = wL^2/8$	359.5	kNm			
Shear capacity of the steel section					
$Vr = \varphi*A_w*f_s$					
Vr ====>	668.44	kN	> Vu ===>	179.75	OK
Moment capacity					
Section selection	406x178x60				
Assume that a temporary lateral restraint is provided at midspan during the construction condition					
Therefore, length between positions of lateral support	4	m			
M_r from red book table 5.5	289	kNm	> Mu ===>	359.50	NOT OK
If it is assumed that temporary lateral restaint is provided					
$M_r = \varphi*Z_{pl}*f_y$ ===>	383.4	kNm	> Mu ===>	359.50	OK
Buckling resistance moment					
Nominal bearing width of hollowcore unit on steel	55	mm			
The length of the beam that may be considered to be fully laterally restrained is 160 x bearing width					
160 x bearing width	8.8	m			
L_{LT}	4	m			
L_{beam}	8	m			
Full lateral restraint is provided			OK		

Figure A.12: Design during construction stage

Composite beam design					
Ultimate limit state loading condition					
Design load, w= (1.2*dead load + 1.6*imposed load)*span	85.44	kN/m			
Design shear, $F_v = wL/2$	341.75	kN			
Design moment, $M_x = wL^2/8$	683.50	kNm			
Shear capacity of the steel section					
$V_r = \varphi*A_w*f_s$					
Same as previously calculated,					
V_r =	668.44	kN	> Vu ===>	341.75	OK
Effective breadth of the slab					
Span/8	1000	mm			
Total infill + gap	1068	mm			
B_{eff} ======>	1000	mm			
Moment capacity					
Section selection					406x178x60
Tensile resistance of steel section, $R_s = \varphi*A*f_y$	2431.4	kN			
Compressive resistance of concrete flange, $R_c=0.45*f_{cu}*B_e*D_s$	3240	kN			
Plastic neutral axis lies within the concrete flange					
The moment resistance of the composite section is dependent on the position of the plastic neutral axis When hollowcore units are used, the plastic neutral axis is not permitted to fall in the concrete slab and must fall within the steel section [SCIP287 pg30] A partial shear connection is thus used to ensure the position of the plastic neutral axis falls within the steel section					
For a partial shear connection					
Characteristic resitance of a shear connector					
19mm diameter, 125mm long in 30MPa concrete					
Q_k	93.5	kN	Red Book shear stud table		
Resistance of a shear connection in sagging moment regions is given by:					
$R_q = N_a*Q_p = 0.8*N_a*Q_k*k$					
k = reduction factor due to hollowcore units = β*ε*sqrt(ω)			SCI P287 4.4.3		
gap between hollowcore units = b_{flange}-2*nominal bearing	67.8	mm			
β - gap width factor	0.98				
Diameter of transverse reinforcement	16	mm			
ε - stud confinement factor	0.9				
ω - transverse joint factor	1.5				
k ====>	1				

Figure A.13: Design during composite stage - Page 1

Design resistance of one shear connector, Q_p	74.8	kN
Number of shear connectors required for full shear transfer:		
R_q = Minimum of R_s and R_c	2431.4	kN
Number of shear connectors required for full shear transfer, N_a	32.5	studs
Therefore need to provide	33.0	studs
Spacing of studs	121.21	mm
R_s=====>	2431.4	kN
R_c=====>	3240	kN
Therefore Rs < Rc		
Degree of shear connection, K = R_s/R_q		
Limit on the degree of shear connection , K > (L-6)/10 ====>	0.2	
1.0 > K > 0.4		

Degree of shear connection , K	R_q [kN]	No. studs	Actual studs (N_a)	Spacing of studs [mm]
80%	1945	26.0	27	148
70%	1702	22.8	23	174
60%	1459	19.5	20	200
50%	1216	16.3	17	235
40%	973	13.0	14	286

B = width of steel flange	177.8	mm
T = Thickness of the steel flange	12.8	mm
Resistance of steel flange, R_f = B * T * f_y	807.9	kN
Resistance of the overall web, R_w = R_s - 2 * R_f	815.5	kN
D	406.4	mm
D_s	240.0	mm

$$M_c = R_s * \frac{D}{2} + R_q * D_s * \left(1 - \frac{R_q}{2 * R_c}\right) - \frac{(R_s - R_q)^2}{R_f} * \frac{T}{4}$$

Degree of shear connection , K	R_q [kN]	Position of NA	Moment resistance - For when PNA is in flange of beam	Mu	
80%	1945.1	Flange	820	683.50	OK
70%	1702.0	Flange	786		OK
60%	1458.8	Flange	757		OK
50%	1215.7	Flange	725		OK
40%	972.6	Flange	689		OK

Therefore the moment resistances for each of the degress of shear connection are sufficient

Due to the geometry of the hollowcore it is ideal to line the shear stud up with the transverse reinforcement. Therefor placing a shear stud every 200mm is ideal

s =	200	mm			
Number of connectors between support and max moment, N_a	20	studs	Total studs over length of beam =	40	studs

Figure A.14: Design during composite stage - Page 2

Transverse Reinforcement					
Diameter of transverse reinforcement	16	mm	Recommended from SCI P287 for when a partial shear connection is used		
For a shear connection of 50%					
Total longitudinal shear force per unit length					
v = N_a*Q_p/s	374	N/mm			
Shear resistance					
v_r = 0.03*A_{cv}*f_{cu} + 0.7*A_{sv}*f_y < 0.8*A_{cv}*sqrt(f_{cu})					
A_{cv}	240	mm^2			
A_{sv} - Shear area for diameter 16mm bar	201.1	mm^2			
Assume reinforcement placed in every second unit, spacing = 200mm					
For shear planes a-a , v====>	187	N/mm			
v_r	394.4	N/mm	< 0.8*A_{cv}*sqrt(f_{cu}) =	1051.62731 N/mm	OK
v_r >	187	N/mm	OK		
For shear planes b-b , v====>	374	N/mm			
v_r	589.91	N/mm	< 0.8*A_{cv}*sqrt(f_{cu}) =	1134.881139 N/mm	OK
v_r >	374		OK		
Therefore using 16mm bars @ 200 centres provides adequate transverse reinforcement					

$$v_r = 0.03 * A_{cv} * f_{cu} + 0.7 * A_{sv} * f_y < 0.8 * A_{cv} * \sqrt{f_{cu}}$$

Figure A.15: Calculation of required transverse reinforcement

Composite beam design		
Serviceabilty limit state loading condition		
$A > D_s^2 * B_{eff}/(D*\alpha_e)$		
A= steel cross sectional area	7610	mm^2
$D_s^2 * B_{eff}/(D*\alpha_e)$ when α = 18	7054.67	mm^2
D - beam depth	406.4	mm
D_s	240	mm
B_{eff}	1000	mm
α_e	13.1	
RHS	10819	
y_g	215	mm
I_g	8.65E+08	mm^4
Serviceability loading condition		
Dead load = 1.1*Gk	5.3	kN/m^2
Imposed load = 1.0*Qk	3.5	kN/m^2
Distributed load on beam	66.1	kN/m
Δ of composite section	20.38	mm
Recommended defelction limits from SCIP287 = L/200	40	mm
Total deflection = defelction before composite action +	48.92	mm
deflection after composite action is attained	20.38	mm
Total deflection ===>	69.30	mm
Deflection is excessive		
Precambering is required		

The second moment of area of the uncracked section is established by transforming the cross sectional area of concrete into an equivalent area of steel by dividing by the modulr ratio, α_e . The composite section may be considered to be uncracked when the equation on the left hand side is satisfied.

$$y_g = \frac{A*\alpha_e*(D+2*D_s)+B_e*D_s^2}{2*(A*\alpha_e+B_e*D_s)}$$

$$I_g = I_x + \frac{B_e*D_s^3}{12*\alpha_e} + \frac{A*B_e*D_s*(D+D_s)^2}{4*(A*\alpha_e+B_e*D_s)}$$

$$\Delta = \frac{5}{384}*\frac{w*L^4}{E*I_{comp}}$$

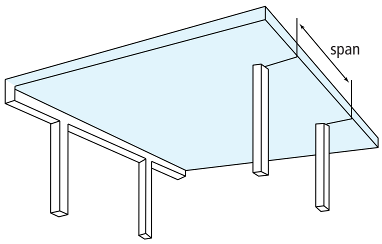
NOT OK

Figure A.16: Serviceability design of composite beams

A.3 Reinforced concrete flat slab structure

Figure A.17 and Figure A.18 illustrate a segment from the design guide that was used to verify structural information regarding the reinforced concrete flat slab structure that was considered.

Flat slabs with edge beams



Introducing edge beams to flat slabs overcomes many of the problems associated with shear at perimeter columns and edge deflection. These slabs are popular for use in office buildings, retail developments, hospitals, hotels, etc. and commonly incorporate upstands rather than downstand perimeter beams. They are economical for spans up to 10 m in square panels.

The chart and data assume an edge loading of 10 kN/m and one 150 mm hole in the slab adjacent to each column. They assume internal columns sizes will at least equal those given in the data. The overall depth of edge beams must be at least 50% greater than the slab thickness.

ADVANTAGES

- Relatively simple and fast formwork and construction
- Architectural finish can be applied directly to the underside of the slab
- Absence of internal beams allows lower storey heights
- Flexibility of partition location and horizontal service distribution.
- Perimeter holes present few problems

DISADVANTAGES

- Perimeter downstand beams may hinder use of table forms

SPAN:DEPTH CHART

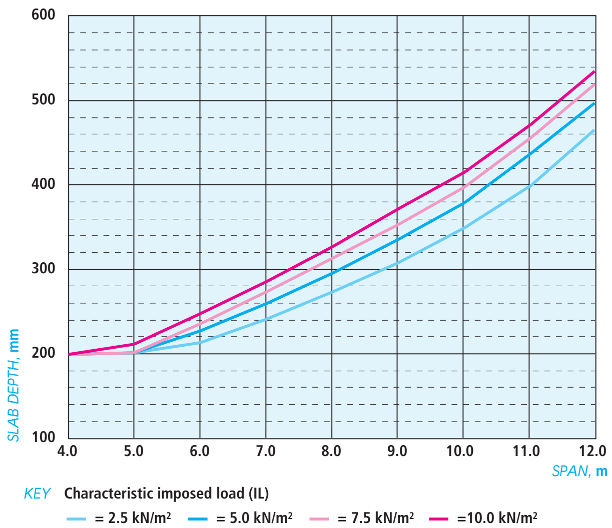


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Figure A.17: Design information for reinforced concrete structure with flat slab floors - Page 1 (Goodchild, 1997)

IN-SITU SLABS

DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS internally and BEAMS around perimeter. Refer to appropriate charts and data to estimate sizes, etc. Minimum column size as data. Edge beams at least 50% deeper than slab.
DIMENSIONS	Square panels, minimum of three spans x three bays. Outside edge flush with columns.
REINFORCEMENT	Main bars: T20 uno. Links R8. To help with deflection, 25% A_sT used as A_s' at midspan of end spans. f_s may have been reduced. 10% allowed for wastage and laps. Beam reinforcement to be added.
LOADS	SDL of 1.50 kN/m² (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads assume elastic reaction factors of 1.0 to internal columns and 0.5 to edge beams.
CONCRETE	C35, 24 kN/m³, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
HOLES	One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m²	200	202	214	242	274	308	350	400	468
IL = 5.0 kN/m²	200	202	228	260	296	336	380	438	500
IL = 7.5 kN/m²	200	202	236	274	314	354	398	456	522
IL = 10.0 kN/m²	200	212	248	286	328	372	416	472	538
ULTIMATE LOAD TO SUPPORTING COLUMNS, internal (edge) per storey, MN									
IL = 2.5 kN/m²	0.2	0.3	0.5	0.7	1.0	1.3	1.8	2.4	3.1
IL = 5.0 kN/m²	0.3	0.4	0.6	0.9	1.3	1.7	2.3	3.0	3.9
IL = 7.5 kN/m²	0.3	0.5	0.8	1.1	1.6	2.1	2.7	3.6	4.6
IL = 10.0 kN/m²	0.4	0.6	1.0	1.4	1.9	2.5	3.2	4.1	5.2
ULTIMATE LOADS ON (EDGE) BEAMS, kN/m — *Includes perimeter load of 10 kN/m but excludes beam self weight*									
IL = 2.5 kN/m²	(28)	(31)	(35)	(40)	(46)	(52)	(61)	(70)	(83)
IL = 5.0 kN/m²	(32)	(36)	(42)	(49)	(56)	(64)	(74)	(86)	(99)
IL = 7.5 kN/m²	(36)	(41)	(49)	(57)	(66)	(76)	(86)	(100)	(115)
IL = 10.0 kN/m²	(40)	(47)	(56)	(65)	(75)	(87)	(99)	(113)	(129)
REINFORCEMENT, kg/m² (kg/m³) — *Beam reinforcement to be added*									
IL = 2.5 kN/m²	8 (39)	10 (50)	15 (70)	18 (75)	22 (80)	26 (85)	30 (85)	35 (87)	40 (85)
IL = 5.0 kN/m²	9 (46)	13 (65)	17 (75)	21 (81)	25 (85)	30 (88)	34 (90)	39 (89)	44 (88)
IL = 7.5 kN/m²	11 (53)	17 (83)	20 (86)	24 (89)	29 (92)	33 (93)	39 (99)	43 (95)	47 (90)
IL = 10.0 kN/m²	12 (61)	19 (87)	22 (91)	27 (97)	33 (101)	39 (104)	44 (106)	48 (101)	52 (96)
COLUMN SIZES ASSUMED, mm square, internal									
IL = 2.5 kN/m²	250	250	260	320	380	440	510	590	680
IL = 5.0 kN/m²	250	250	310	370	430	500	580	660	750
IL = 7.5 kN/m²	250	280	340	410	480	550	630	720	820
IL = 10.0 kN/m²	250	300	370	440	520	600	680	770	870
DESIGN NOTES a = q_k > 1.25 g_k b = q_k > 5 kN/m² f = shear critical (initially v>2v_c) g = T25s used h = T32s used									
IL = 2.5 kN/m²							g	g	h
IL = 5.0 kN/m²						g	g	h	h
IL = 7.5 kN/m²	b	b	b	b	bg	bg	bh	bh	bh
IL = 10.0 kN/m²	ab	ab	ab	b	bfg	bfh	bfh	bfh	bh
LINKS, MAXIMUM NUMBER OF PERIMETERS (and percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m²	0 (0.0%)	2 (0.7%)	3 (0.7%)	3 (0.6%)	4 (0.9%)	4 (1.0%)	4 (1.0%)	4 (1.1%)	4 (1.1%)
IL = 5.0 kN/m²	2 (1.2%)	3 (1.0%)	3 (0.9%)	4 (1.2%)	4 (1.2%)	4 (1.2%)	4 (1.3%)	4 (1.2%)	5 (1.4%)
IL = 7.5 kN/m²	3 (1.8%)	4 (1.5%)	4 (1.3%)	4 (1.3%)	4 (1.4%)	4 (1.4%)	4 (1.3%)	5 (1.6%)	5 (1.6%)
IL = 10.0 kN/m²	3 (1.6%)	4 (1.7%)	4 (1.5%)	4 (1.6%)	5 (1.7%)	6 (2.0%)	5 (1.6%)	5 (1.6%)	5 (1.8%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Fire resistance	2 hours	+5 mm	4 hours	+30 mm
Exposure	Moderate	+20 mm	Severe, C40 concrete	+25 mm
Other	300 square holes	+0 mm	50 mm drops, L/3 wide	-5 mm
	Using T25s cf T20s	+5 mm	2 spans	+ 0 mm

Thickness, mm	**Span, m**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
	No shear links	242	308	394	460	498	554	640

Rectangular panels: equivalent spans, m — Use an equivalent square span, below, to derive thickness

Long span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Short span = 5.0 m	5.1	5.7	6.4	7.2	7.9	8.8	
Short span = 6.0 m	6.0	6.3	6.9	7.5	8.2	9.2	10.0
Short span = 7.0 m		7.0	7.4	8.0	8.6	9.4	10.2
Short span = 8.0 m			8.0	8.5	9.0	9.6	10.4
Short span = 9.0 m				9.0	9.3	9.9	10.7
Short span =10.0 m					10.0	10.4	11.0
Short span =11.0 m						11.0	11.4

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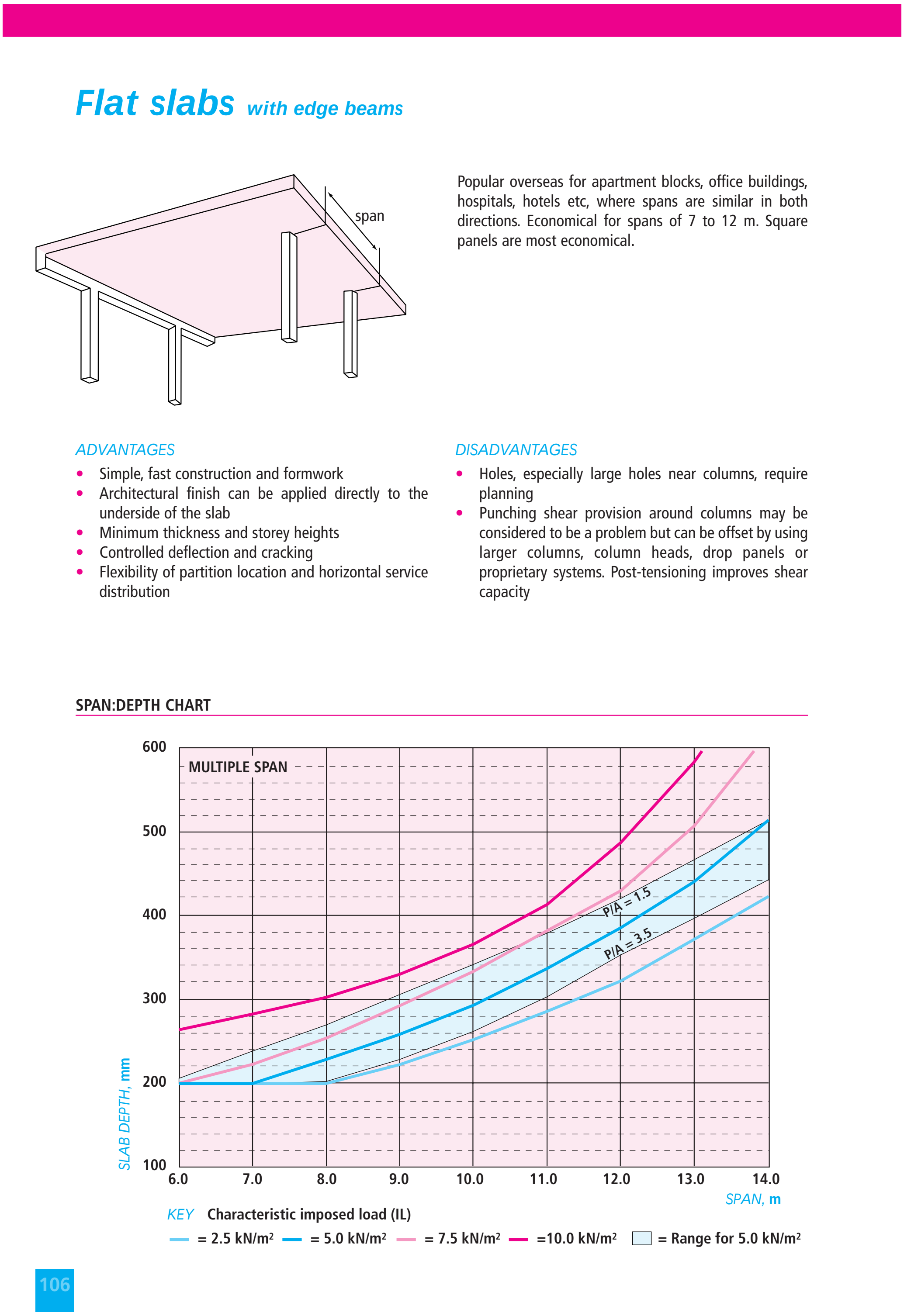
Figure A.18: Design information for reinforced concrete structure with flat slab floors - Page 1 (Goodchild, 1997)

A.4 Reinforced concrete structure with post-tensioned floors

Figure A.19 and Figure A.20 illustrate a segment from the design guide that was used to verify structural information regarding the post-tensioned concrete flat slab structure that was considered.



Flat slabs *with edge beams*

Popular overseas for apartment blocks, office buildings, hospitals, hotels etc, where spans are similar in both directions. Economical for spans of 7 to 12 m. Square panels are most economical.

ADVANTAGES

- Simple, fast construction and formwork
- Architectural finish can be applied directly to the underside of the slab
- Minimum thickness and storey heights
- Controlled deflection and cracking
- Flexibility of partition location and horizontal service distribution

DISADVANTAGES

- Holes, especially large holes near columns, require planning
- Punching shear provision around columns may be considered to be a problem but can be offset by using larger columns, column heads, drop panels or proprietary systems. Post-tensioning improves shear capacity

SPAN:DEPTH CHART

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Figure A.19: Design information for reinforced concrete structure with flat slab floors - Page 1 (Goodchild, 1997)

POST-TENSIONED SLABS

DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS internally and BEAMS around perimeter. Refer to charts and data to estimate sizes, etc.
DESIGN BASIS	To CS TR43. Balanced load 133% DL + 33% IL . Maximum prestress (P/A) = 2.5 N/mm^2. See Section 7. Effectively Class 2 assumed. No restraint to movement assumed.
DIMENSIONS	Square panels, assuming three spans by three bays. Outside edge flush with columns. Minimum column size as data. Edge beams should be at least 50% deeper than slab.
LOADS	SDL of 1.50 kN/m^2 (finishes) assumed. Perimeter load of 10 kN/m (14 kN/m ult) included in loads on edge beams. Ultimate loads to columns and beams are the result of moment distribution analysis.
TENDONS	Unbonded 15.7 mm diam. Superstrand (A_{ps} 150 mm^2, f_{pu} 1770 N/mm^2), B1,T2, B2 & T3. Max 5 per m.
CONCRETE	C40, 24 kN/m^3, 20 mm aggregate. f_{ci} = 25 N/mm^2.
REINFORCEMENT	Assumed min. T10@250T both ways at supports, min T12@500B both ways and T8 links. 10% allowed for wastage and laps.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	200	200	200	222	252	286	322	372	424
IL = 5.0 kN/m^2	200	200	228	258	294	338	386	442	516
IL = 7.5 kN/m^2	200	222	254	292	334	382	430	508	620
IL = 10.0 kN/m^2	264	282	302	330	366	414	488	584	710
ULTIMATE LOAD TO SUPPORTING COLUMNS, MN, INTERNAL, PER STOREY.									
IL = 2.5 kN/m^2	0.61	0.83	1.09	1.45	1.93	2.49	3.22	4.16	5.27
IL = 5.0 kN/m^2	0.80	1.09	1.50	2.01	2.62	3.43	4.39	5.58	7.11
IL = 7.5 kN/m^2	0.99	1.40	1.92	2.55	3.35	4.31	5.44	6.97	9.05
IL = 10.0 kN/m^2	1.28	1.79	2.39	3.11	4.02	5.12	6.57	8.43	10.88
ULTIMATE LOADS ON EDGE BEAMS, kN/m									
IL = 2.5 kN/m^2	48	54	59	68	78	89	102	119	137
IL = 5.0 kN/m^2	59	66	77	89	101	118	135	155	181
IL = 7.5 kN/m^2	70	81	94	109	126	144	164	191	227
IL = 10.0 kN/m^2	86	100	114	130	148	169	196	229	270
REINFORCEMENT (TENDONS), kg/m^2									
IL = 2.5 kN/m^2	14 (8)	13 (9)	13 (9)	12 (10)	12 (11)	11 (12)	11 (13)	12 (13)	14 (13)
IL = 5.0 kN/m^2	14 (8)	14 (9)	13 (10)	13 (11)	13 (13)	13 (13)	14 (13)	16 (13)	18 (13)
IL = 7.5 kN/m^2	15 (8)	14 (10)	14 (11)	13 (13)	13 (13)	14 (13)	17 (13)	19 (13)	21 (13)
IL = 10.0 kN/m^2	15 (7)	15 (9)	15 (12)	15 (13)	16 (13)	18 (13)	19 (13)	22 (13)	24 (13)
COLUMN SIZES ASSUMED, INTERNAL, mm square,									
IL = 2.5 kN/m^2	280	330	380	440	500	570	650	740	830
IL = 5.0 kN/m^2	320	370	440	510	580	660	750	840	950
IL = 7.5 kN/m^2	350	410	480	560	640	730	820	920	1050
IL = 10.0 kN/m^2	390	460	530	610	690	780	880	1000	1130
DESIGN NOTES o = limited by P/A of 2.5 N/mm^2; p = 8 > response factor > 4; q = shrinkage per span > 10 mm; r = tendons @ < 300 mm cc. (R @ 200 mm cc.); s = overall deflection, d_{xx} + d_{yy}, > 20 mm (S > 30 mm)									
IL = 2.5 kN/m^2	p	o	os	ors	orS	orS	oRS	RS	RS
IL = 5.0 kN/m^2	p	o	or	ors	ors	Rs	Rs	Rs	RS
IL = 7.5 kN/m^2	p	or	or	or	R	Rs	Rs	Rs	Rs
IL = 10.0 kN/m^2			r	r	R	Rs	R	R	R
LINKS, maximum number of perimeters (and percentage by weight of bonded reinforcement), no. (%)									
IL = 2.5 kN/m^2	2 (0.6%)	3 (0.8%)	5 (1.4%)	6 (1.6%)	7 (1.8%)	7 (1.6%)	8 (1.9%)	8 (1.7%)	7 (1.1%)
IL = 5.0 kN/m^2	3 (1.0%)	5 (1.7%)	6 (1.9%)	7 (2.0%)	7 (1.8%)	8 (2.1%)	8 (1.8%)	7 (1.1%)	6 (0.7%)
IL = 7.5 kN/m^2	4 (1.5%)	6 (2.2%)	7 (2.4%)	7 (2.1%)	8 (2.3%)	8 (1.9%)	7 (1.2%)	6 (0.8%)	5 (0.5%)
IL = 10.0 kN/m^2	3 (1.1%)	5 (1.7%)	6 (2.0%)	7 (2.1%)	7 (1.7%)	7 (1.4%)	6 (0.9%)	5 (0.6%)	4 (0.4%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours	+0 mm	4 hours	+25 mm
Exposure	Moderate	+5 mm	Severe	+15 mm
Serviceability	Class 1	n/a	Column heads L/10 wide	-0 mm
Two spans	2 spans by 3 bays	see below	2 spans by 2 bays	see below
Rectangular bays	6.0 m wide bay	-15 mm @ 8 m and beyond	9.0 m wide bay	-15 mm @ 11.0 m and beyond

Thickness, mm	**Spans, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 1.5 N/mm^2 max	270	306	342	380	422	468	516
	P/A 3.5 N/mm^2 max #	202	228	262	304	354	398	444
	2 spans by 3 bays	230	260	294	346	400	496	608
	2 spans by 2 bays	238	268	300	356	430	524	636
	T16@350B both ways	220	246	274	306	360	424	516

max 7 tendons/m

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Figure A.20: Design information for reinforced concrete structure with flat slab floors - Page 1 (Goodchild, 1997)

A.5 Foundation calculations

The size of the foundations was calculated for each of the structural alternatives that were considered. The size of the foundations that were required were calculated with the base designer software in Prokon. An example of the output from the Prokon base designer software is shown in Figure A.21 and Figure A.22.

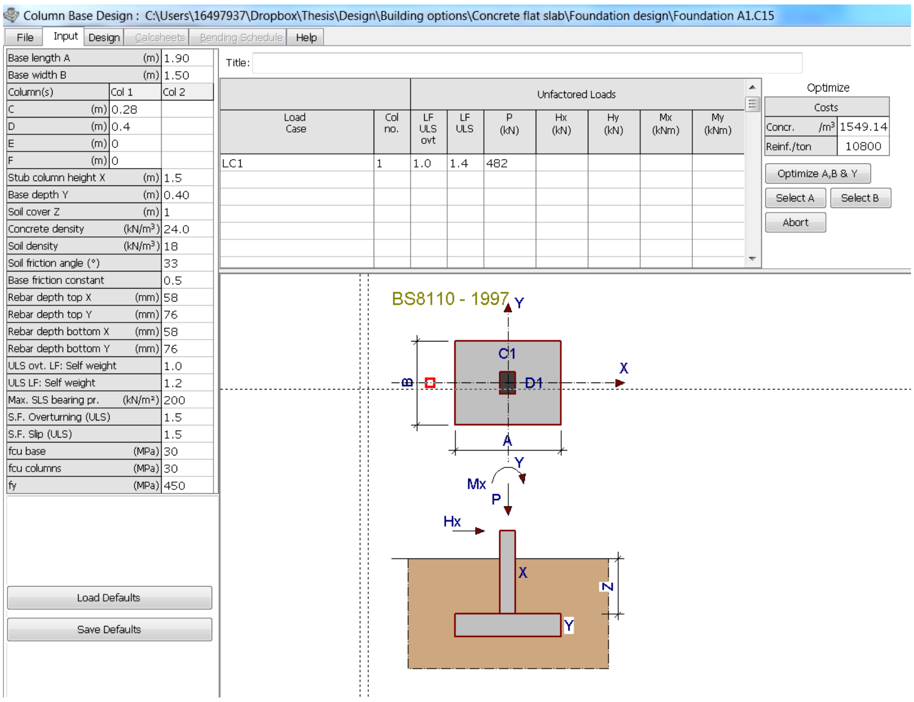


Figure A.21: Example of Prokon output for calculating size of pad footings - Input page

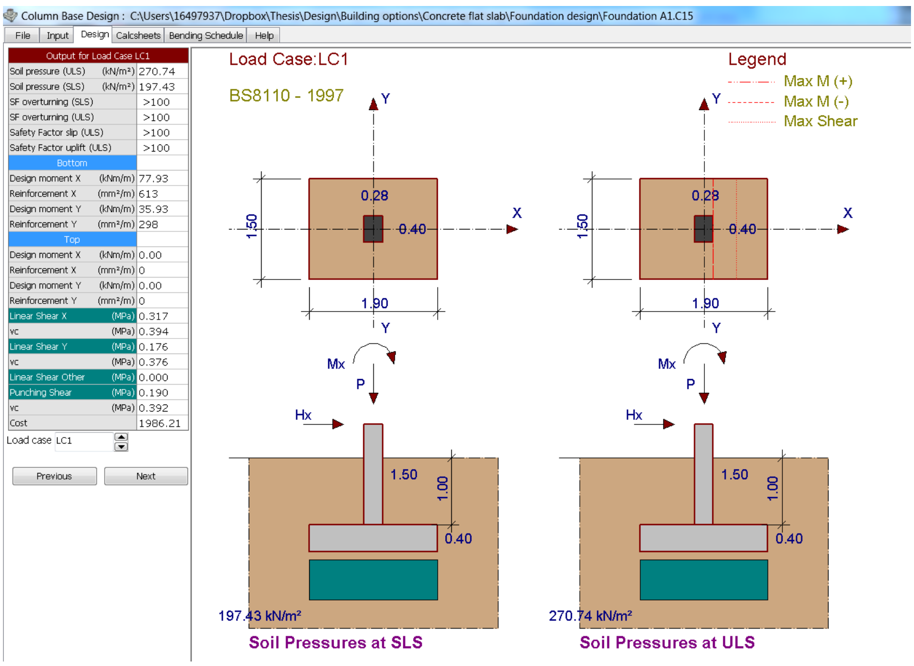


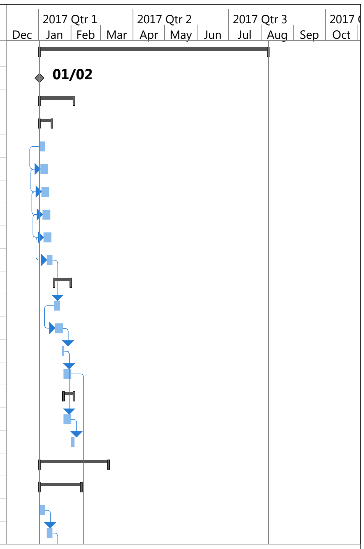
Figure A.22: Example of Prokon output for calculating size of pad footings - Results

Appendix B

Detailed construction programmes

Additional information is provided in this appendix regarding the construction programmes developed in this study. A detailed construction programme has been included for both the steel composite and RC flat slab structures. All preceding and succeeding trades have been indicated. The programmes were prepared with assistance from project management specialists in construction projects. This ensured that the items that have been included can be seen as being representative of a typical multi-storey office building, and that time durations are realistic.

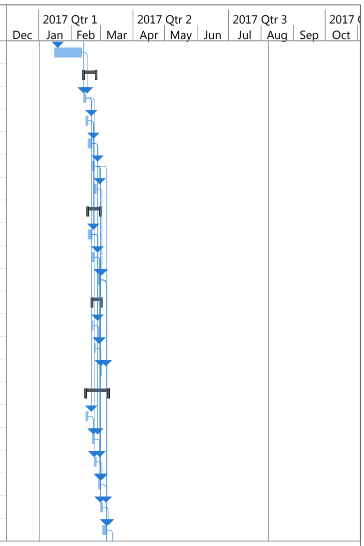
ID	Task Name	Duration	Start	Finish
1	**STEEL COMPOSITE STRUCTURE**	**156 days**	**Mon 17/01/02**	**Mon 17/08/07**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
4	**RC bases**	**10 days**	**Mon 17/01/02**	**Fri 17/01/13**
5	Excavate / trim bases	5 days	Mon 17/01/02	Fri 17/01/06
6	Concrete blinding	5 days	Tue 17/01/03	Mon 17/01/09
7	Reinforcing to bases	5 days	Wed 17/01/04	Tue 17/01/10
8	Shutter bases	5 days	Thu 17/01/05	Wed 17/01/11
9	Concrete to bases	5 days	Fri 17/01/06	Thu 17/01/12
10	Backfilling	5 days	Mon 17/01/09	Fri 17/01/13
11	**Stub columns**	**12 days**	**Mon 17/01/16**	**Tue 17/01/31**
12	Rebar to stub columns	5 days	Mon 17/01/16	Fri 17/01/20
13	Shutter to stub columns, including HD bolts	5 days	Tue 17/01/17	Mon 17/01/23
14	Concrete to GF stub columns	1 day	Tue 17/01/24	Tue 17/01/24
15	Cure stub columns	5 days	Wed 17/01/25	Tue 17/01/31
16	**Surface bed**	**8 days**	**Wed 17/01/25**	**Fri 17/02/03**
17	Under slab services	5 days	Wed 17/01/25	Tue 17/01/31
18	Prepare for/cast surface bed and powerfloat	3 days	Wed 17/02/01	Fri 17/02/03
19	**Steel frame and composite floors**	**48 days**	**Mon 17/01/02**	**Wed 17/03/08**
20	**Lead in time for steel work**	**30 days**	**Mon 17/01/02**	**Fri 17/02/10**
21	Prepare drawings	5 days	Mon 17/01/02	Fri 17/01/06
22	Submission and approval	5 days	Mon 17/01/09	Fri 17/01/13



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

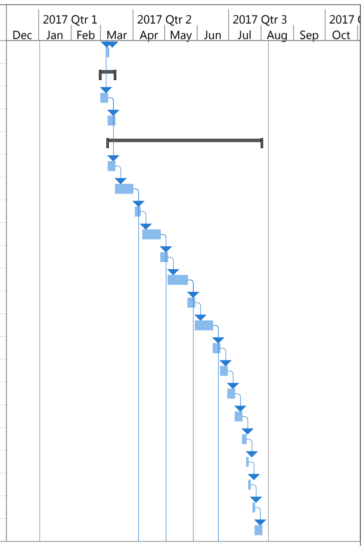
ID	Task Name	Duration	Start	Finish
23	Order, fabricate and deliver	20 days	Mon 17/01/16	Fri 17/02/10
24	**Erect steel frame**	**10 days**	**Mon 17/02/13**	**Fri 17/02/24**
25	Erect columns GF - 2F	2 days	Mon 17/02/13	Tue 17/02/14
26	Steel beams 1F	2 days	Wed 17/02/15	Thu 17/02/16
27	Steel beams 2F	2 days	Fri 17/02/17	Mon 17/02/20
28	Erect columns 2F - Roof	2 days	Tue 17/02/21	Wed 17/02/22
29	Steel beams 3F	2 days	Thu 17/02/23	Fri 17/02/24
30	**Bondek sheet and place reinforcement**	**8 days**	**Fri 17/02/17**	**Tue 17/02/28**
31	Lift sheets to 1st floor and place reinforcement	2 days	Fri 17/02/17	Mon 17/02/20
32	Lift and place sheets 2F and place reinforcemen	2 days	Tue 17/02/21	Wed 17/02/22
33	Lift and place sheets 3F and place reinforcemen	2 days	Mon 17/02/27	Tue 17/02/28
34	**Concrete pours**	**7 days**	**Tue 17/02/21**	**Wed 17/03/01**
35	Pour concrete to 1st floor	1 day	Tue 17/02/21	Tue 17/02/21
36	Pour concrete to 2nd floor	1 day	Thu 17/02/23	Thu 17/02/23
37	Pour concrete to 3rd floor	1 day	Wed 17/03/01	Wed 17/03/01
38	**On-site application of vermiculite spray**	**16 days**	**Wed 17/02/15**	**Wed 17/03/08**
39	Ground floor columns	2 days	Wed 17/02/15	Thu 17/02/16
40	1st floor beams	2 days	Tue 17/02/21	Wed 17/02/22
41	1st floor columns	2 days	Thu 17/02/23	Fri 17/02/24
42	2nd floor beams	2 days	Mon 17/02/27	Tue 17/02/28
43	2nd floor columns	2 days	Wed 17/03/01	Thu 17/03/02
44	3rd floor beams	2 days	Fri 17/03/03	Mon 17/03/06



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

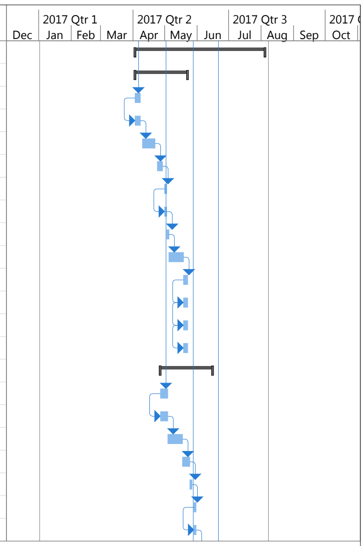
ID	Task Name	Duration	Start	Finish
45	3rd floor columns	2 days	Tue 17/03/07	Wed 17/03/08
46	**Roof works**	**10 days**	**Wed 17/03/01**	**Tue 17/03/14**
47	Roof steel structure	5 days	Wed 17/03/01	Tue 17/03/07
48	Insulation /roof sheeting	5 days	Wed 17/03/08	Tue 17/03/14
49	**Cladding**	**105 days**	**Wed 17/03/08**	**Tue 17/08/01**
50	Scaffolding G-1	5 days	Wed 17/03/08	Tue 17/03/14
51	Light steel framing G-1	13 days	Wed 17/03/15	Fri 17/03/31
52	Scaffolding 1-2	5 days	Mon 17/04/03	Fri 17/04/07
53	Light steel framing 1-2	13 days	Mon 17/04/10	Wed 17/04/26
54	Scaffolding 2-3	5 days	Thu 17/04/27	Wed 17/05/03
55	Light steel framing 2-3	13 days	Thu 17/05/04	Mon 17/05/22
56	Scaffolding 3-4	5 days	Tue 17/05/23	Mon 17/05/29
57	Light steel framing 3-4	13 days	Tue 17/05/30	Thu 17/06/15
58	Plastering G-1	5 days	Fri 17/06/16	Thu 17/06/22
59	Plastering 1-2	5 days	Fri 17/06/23	Thu 17/06/29
60	Plastering 2-3	5 days	Fri 17/06/30	Thu 17/07/06
61	Plastering 3-4	5 days	Fri 17/07/07	Thu 17/07/13
62	Painting G-1	2 days	Fri 17/07/14	Mon 17/07/17
63	Painting 1-2	2 days	Tue 17/07/18	Wed 17/07/19
64	Painting 2-3	2 days	Thu 17/07/20	Fri 17/07/21
65	Painting 3-4	2 days	Mon 17/07/24	Tue 17/07/25
66	Remove scaffolding	5 days	Wed 17/07/26	Tue 17/08/01



Project: Construction programm
Date: Wed 16/10/19

Task		Inactive Summary		External Tasks	
Split		Manual Task		External Milestone	
Milestone		Duration-only		Deadline	
Summary		Manual Summary Rollup		Progress	
Project Summary		Manual Summary		Manual Progress	
Inactive Task		Start-only			
Inactive Milestone		Finish-only			

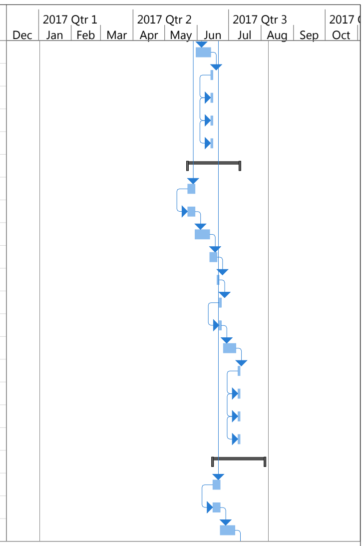
ID	Task Name	Duration	Start	Finish
67	**Services and finishes**	**90 days**	**Mon 17/04/03**	**Fri 17/08/04**
68	**Ground floor**	**36 days**	**Mon 17/04/03**	**Mon 17/05/22**
69	1st fix sprinklers	5 days	Mon 17/04/03	Fri 17/04/07
70	1st fix AC	5 days	Mon 17/04/03	Fri 17/04/07
71	Internal walls (Drywall)	10 days	Mon 17/04/10	Fri 17/04/21
72	Ceilings	5 days	Mon 17/04/24	Fri 17/04/28
73	Doors and glazing	2 days	Mon 17/05/01	Tue 17/05/02
74	Floor finishes	2 days	Mon 17/05/01	Tue 17/05/02
75	Wall finishes	2 days	Wed 17/05/03	Thu 17/05/04
76	Cabinetry	10 days	Fri 17/05/05	Thu 17/05/18
77	2nd fix electrical	2 days	Fri 17/05/19	Mon 17/05/22
78	2nd fix plumbing	2 days	Fri 17/05/19	Mon 17/05/22
79	2nd fix AC	2 days	Fri 17/05/19	Mon 17/05/22
80	2nd fix sprinklers	2 days	Fri 17/05/19	Mon 17/05/22
81	**1st floor**	**36 days**	**Thu 17/04/27**	**Thu 17/06/15**
82	1st fix sprinklers	5 days	Thu 17/04/27	Wed 17/05/03
83	1st fix AC	5 days	Thu 17/04/27	Wed 17/05/03
84	Internal walls (Drywall)	10 days	Thu 17/05/04	Wed 17/05/17
85	Ceilings	5 days	Thu 17/05/18	Wed 17/05/24
86	Doors and glazing	2 days	Thu 17/05/25	Fri 17/05/26
87	Floor finishes	2 days	Mon 17/05/29	Tue 17/05/30
88	Wall finishes	2 days	Mon 17/05/29	Tue 17/05/30



Project: Construction programm
Date: Wed 16/10/19

Task		Inactive Summary		External Tasks	
Split		Manual Task		External Milestone	
Milestone		Duration-only		Deadline	
Summary		Manual Summary Rollup		Progress	
Project Summary		Manual Summary		Manual Progress	
Inactive Task		Start-only			
Inactive Milestone		Finish-only			

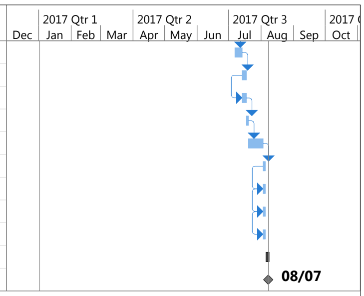
ID	Task Name	Duration	Start	Finish
89	Cabinetry	10 days	Wed 17/05/31	Tue 17/06/13
90	2nd fix electrical	2 days	Wed 17/06/14	Thu 17/06/15
91	2nd fix plumbing	2 days	Wed 17/06/14	Thu 17/06/15
92	2nd fix AC	2 days	Wed 17/06/14	Thu 17/06/15
93	2nd fix sprinklers	2 days	Wed 17/06/14	Thu 17/06/15
94	**2nd floor**	**36 days**	**Tue 17/05/23**	**Tue 17/07/11**
95	1st fix sprinklers	5 days	Tue 17/05/23	Mon 17/05/29
96	1st fix AC	5 days	Tue 17/05/23	Mon 17/05/29
97	Internal walls (Drywall)	10 days	Tue 17/05/30	Mon 17/06/12
98	Ceilings	5 days	Tue 17/06/13	Mon 17/06/19
99	Doors and glazing	2 days	Tue 17/06/20	Wed 17/06/21
100	Floor finishes	2 days	Thu 17/06/22	Fri 17/06/23
101	Wall finishes	2 days	Thu 17/06/22	Fri 17/06/23
102	Cabinetry	10 days	Mon 17/06/26	Fri 17/07/07
103	2nd fix electrical	2 days	Mon 17/07/10	Tue 17/07/11
104	2nd fix plumbing	2 days	Mon 17/07/10	Tue 17/07/11
105	2nd fix AC	2 days	Mon 17/07/10	Tue 17/07/11
106	2nd fix sprinklers	2 days	Mon 17/07/10	Tue 17/07/11
107	**3rd floor**	**36 days**	**Fri 17/06/16**	**Fri 17/08/04**
108	1st fix sprinklers	5 days	Fri 17/06/16	Thu 17/06/22
109	1st fix AC	5 days	Fri 17/06/16	Thu 17/06/22
110	Internal walls (Drywall)	10 days	Fri 17/06/23	Thu 17/07/06



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

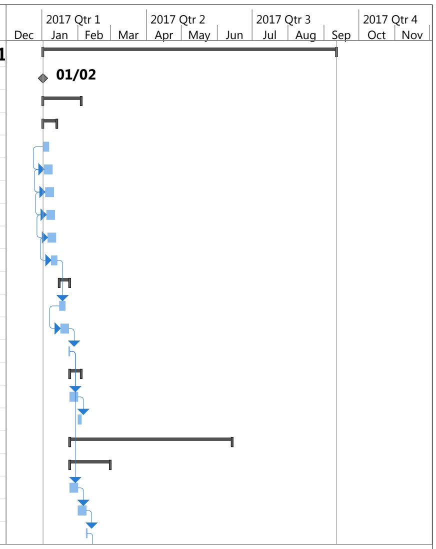
ID	Task Name	Duration	Start	Finish
111	Ceilings	5 days	Fri 17/07/07	Thu 17/07/13
112	Doors and glazing	2 days	Fri 17/07/14	Mon 17/07/17
113	Floor finishes	2 days	Fri 17/07/14	Mon 17/07/17
114	Wall finishes	2 days	Tue 17/07/18	Wed 17/07/19
115	Cabinetry	10 days	Thu 17/07/20	Wed 17/08/02
116	2nd fix electrical	2 days	Thu 17/08/03	Fri 17/08/04
117	2nd fix plumbing	2 days	Thu 17/08/03	Fri 17/08/04
118	2nd fix AC	2 days	Thu 17/08/03	Fri 17/08/04
119	2nd fix sprinklers	2 days	Thu 17/08/03	Fri 17/08/04
120	**Close out Phase**	**1 day**	**Mon 17/08/07**	**Mon 17/08/07**
122	Final handover	0 days	Mon 17/08/07	Mon 17/08/07



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

ID	Task Name	Duration	Start	Finish
1	**RC FLAT SLAB STRUCTURE**	**181 days**	**Mon 17/01/02**	**Mon 17/09/11**
2	Site establishment	0 days	Mon 17/01/02	Mon 17/01/02
3	**Foundations and building substructure**	**25 days**	**Mon 17/01/02**	**Fri 17/02/03**
4	**RC bases**	**10 days**	**Mon 17/01/02**	**Fri 17/01/13**
5	Excavate / trim bases	5 days	Mon 17/01/02	Fri 17/01/06
6	Concrete blinding	5 days	Tue 17/01/03	Mon 17/01/09
7	Reinforcing to bases	5 days	Wed 17/01/04	Tue 17/01/10
8	Shutter bases	5 days	Thu 17/01/05	Wed 17/01/11
9	Concrete to bases	5 days	Fri 17/01/06	Thu 17/01/12
10	Backfilling	5 days	Mon 17/01/09	Fri 17/01/13
11	**Stub columns**	**7 days**	**Mon 17/01/16**	**Tue 17/01/24**
12	Shutters to GF stub columns	5 days	Mon 17/01/16	Fri 17/01/20
13	Reinforcing to GF stub columns	5 days	Tue 17/01/17	Mon 17/01/23
14	Concrete to GF stub columns	1 day	Tue 17/01/24	Tue 17/01/24
15	**Surface bed**	**8 days**	**Wed 17/01/25**	**Fri 17/02/03**
16	Under slab services	5 days	Wed 17/01/25	Tue 17/01/31
17	Prepare for/cast surface bed and powerf	3 days	Wed 17/02/01	Fri 17/02/03
18	**Reinforced concrete frame**	**100 days**	**Wed 17/01/25**	**Tue 17/06/13**
19	**Ground floor columns and first floor slab**	**25 days**	**Wed 17/01/25**	**Tue 17/02/28**
20	Shutters to GF columns	5 days	Wed 17/01/25	Tue 17/01/31
21	Reinforcing to GF columns	5 days	Wed 17/02/01	Tue 17/02/07
22	RC concrete to GF columns	1 day	Wed 17/02/08	Wed 17/02/08



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

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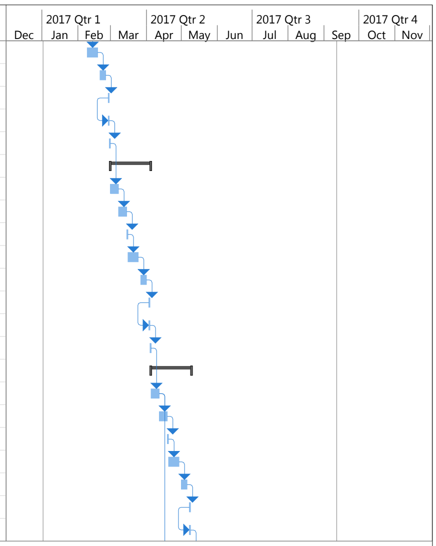
ID	Task Name	Duration	Start	Finish
23	Staging for 1st floor slab	7 days	Thu 17/02/09	Fri 17/02/17
24	Reinforcing to 1st floor slab	5 days	Mon 17/02/20	Fri 17/02/24
25	1st fix electrical	1 day	Mon 17/02/27	Mon 17/02/27
26	1st fix plumbing	1 day	Mon 17/02/27	Mon 17/02/27
27	RC to 1st floor	1 day	Tue 17/02/28	Tue 17/02/28
28	**First floor columns and second floor slab**	**25 days**	**Wed 17/03/01**	**Tue 17/04/04**
29	Shutters to 1st floor columns	5 days	Wed 17/03/01	Tue 17/03/07
30	Reinforcing to 1st floor columns	5 days	Wed 17/03/08	Tue 17/03/14
31	RC concrete to 1st floor columns	1 day	Wed 17/03/15	Wed 17/03/15
32	Staging for 2nd floor slab	7 days	Thu 17/03/16	Fri 17/03/24
33	Reinforcing to 2nd floor slab	5 days	Mon 17/03/27	Fri 17/03/31
34	1st fix electrical	1 day	Mon 17/04/03	Mon 17/04/03
35	1st fix plumbing	1 day	Mon 17/04/03	Mon 17/04/03
36	RC to 2nd floor	1 day	Tue 17/04/04	Tue 17/04/04
37	**Second floor columns and third floor slab**	**25 days**	**Wed 17/04/05**	**Tue 17/05/09**
38	Shutters to 2nd floor columns	5 days	Wed 17/04/05	Tue 17/04/11
39	Reinforcing to 2nd floor columns	5 days	Wed 17/04/12	Tue 17/04/18
40	RC concrete to 2nd floor columns	1 day	Wed 17/04/19	Wed 17/04/19
41	Staging for 3rd floor slab	7 days	Thu 17/04/20	Fri 17/04/28
42	Reinforcing to 3rd floor slab	5 days	Mon 17/05/01	Fri 17/05/05
43	1st fix electrical	1 day	Mon 17/05/08	Mon 17/05/08
44	1st fix plumbing	1 day	Mon 17/05/08	Mon 17/05/08



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

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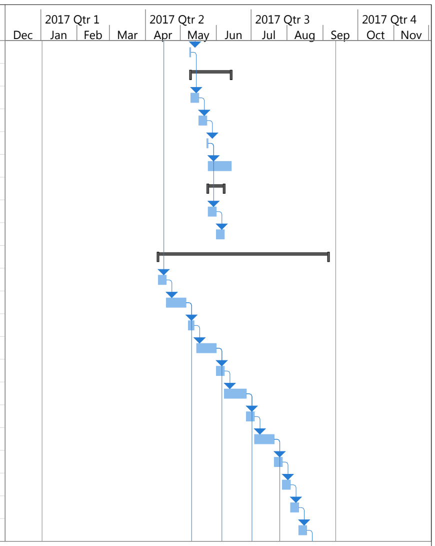
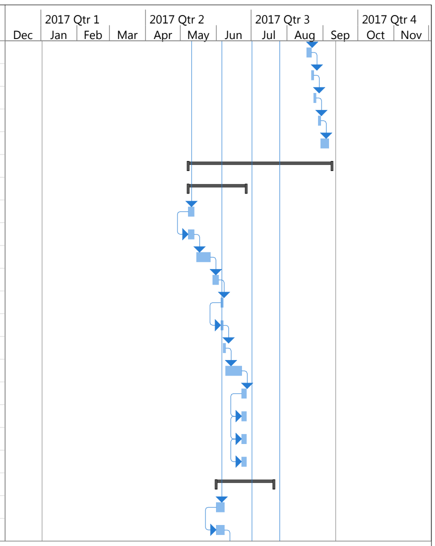
ID	Task Name	Duration	Start	Finish
45	RC to 3rd floor	1 day	Tue 17/05/09	Tue 17/05/09
46	**Third floor columns to roof only**	**25 days**	**Wed 17/05/10**	**Tue 17/06/13**
47	Shutters to 3rd floor columns	5 days	Wed 17/05/10	Tue 17/05/16
48	Reinforcing to 3rd floor columns	5 days	Wed 17/05/17	Tue 17/05/23
49	RC concrete to 3rd floor columns	1 day	Wed 17/05/24	Wed 17/05/24
50	Remove shutters	14 days	Thu 17/05/25	Tue 17/06/13
51	**Roof works**	**10 days**	**Thu 17/05/25**	**Wed 17/06/07**
52	Roof steel structure	5 days	Thu 17/05/25	Wed 17/05/31
53	Insulation /roof sheeting	5 days	Thu 17/06/01	Wed 17/06/07
54	**Cladding**	**105 days**	**Wed 17/04/12**	**Tue 17/09/05**
55	Scaffolding G-1	5 days	Wed 17/04/12	Tue 17/04/18
56	Brick and mortar cladding	13 days	Wed 17/04/19	Fri 17/05/05
57	Scaffolding 1-2	5 days	Mon 17/05/08	Fri 17/05/12
58	Brick and mortar cladding	13 days	Mon 17/05/15	Wed 17/05/31
59	Scaffolding 2-3	5 days	Thu 17/06/01	Wed 17/06/07
60	Brick and mortar cladding	13 days	Thu 17/06/08	Mon 17/06/26
61	Scaffolding 3-4	5 days	Tue 17/06/27	Mon 17/07/03
62	Brick and mortar cladding	13 days	Tue 17/07/04	Thu 17/07/20
63	Plastering G-1	5 days	Fri 17/07/21	Thu 17/07/27
64	Plastering 1-2	5 days	Fri 17/07/28	Thu 17/08/03
65	Plastering 2-3	5 days	Fri 17/08/04	Thu 17/08/10
66	Plastering 3-4	5 days	Fri 17/08/11	Thu 17/08/17



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

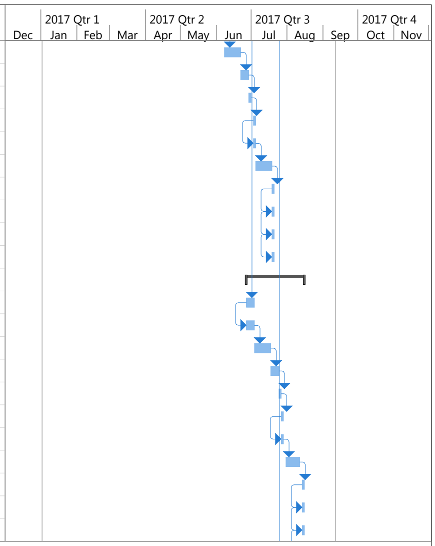
ID	Task Name	Duration	Start	Finish
67	Painting G-1	2 days	Fri 17/08/18	Mon 17/08/21
68	Painting 1-2	2 days	Tue 17/08/22	Wed 17/08/23
69	Painting 2-3	2 days	Thu 17/08/24	Fri 17/08/25
70	Painting 3-4	2 days	Mon 17/08/28	Tue 17/08/29
71	Remove scaffolding	5 days	Wed 17/08/30	Tue 17/09/05
72	**Services and finishes**	**90 days**	**Mon 17/05/08**	**Fri 17/09/08**
73	**Ground floor**	**36 days**	**Mon 17/05/08**	**Mon 17/06/26**
74	1st fix sprinklers	5 days	Mon 17/05/08	Fri 17/05/12
75	1st fix AC	5 days	Mon 17/05/08	Fri 17/05/12
76	Internal walls (Drywall)	10 days	Mon 17/05/15	Fri 17/05/26
77	Ceilings	5 days	Mon 17/05/29	Fri 17/06/02
78	Doors and glazing	2 days	Mon 17/06/05	Tue 17/06/06
79	Floor finishes	2 days	Mon 17/06/05	Tue 17/06/06
80	Wall finishes	2 days	Wed 17/06/07	Thu 17/06/08
81	Cabinetry	10 days	Fri 17/06/09	Thu 17/06/22
82	2nd fix electrical	2 days	Fri 17/06/23	Mon 17/06/26
83	2nd fix plumbing	2 days	Fri 17/06/23	Mon 17/06/26
84	2nd fix AC	2 days	Fri 17/06/23	Mon 17/06/26
85	2nd fix sprinklers	2 days	Fri 17/06/23	Mon 17/06/26
86	**1st floor**	**36 days**	**Thu 17/06/01**	**Thu 17/07/20**
87	1st fix sprinklers	5 days	Thu 17/06/01	Wed 17/06/07
88	1st fix AC	5 days	Thu 17/06/01	Wed 17/06/07



Project: Construction programm
Date: Wed 16/10/19

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone
Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only
External Tasks | External Milestone | Deadline | Progress | Manual Progress

ID	Task Name	Duration	Start	Finish
89	Internal walls (Drywall)	10 days	Thu 17/06/08	Wed 17/06/21
90	Ceilings	5 days	Thu 17/06/22	Wed 17/06/28
91	Doors and glazing	2 days	Thu 17/06/29	Fri 17/06/30
92	Floor finishes	2 days	Mon 17/07/03	Tue 17/07/04
93	Wall finishes	2 days	Mon 17/07/03	Tue 17/07/04
94	Cabinetry	10 days	Wed 17/07/05	Tue 17/07/18
95	2nd fix electrical	2 days	Wed 17/07/19	Thu 17/07/20
96	2nd fix plumbing	2 days	Wed 17/07/19	Thu 17/07/20
97	2nd fix AC	2 days	Wed 17/07/19	Thu 17/07/20
98	2nd fix sprinklers	2 days	Wed 17/07/19	Thu 17/07/20
99	**2nd floor**	**36 days**	**Tue 17/06/27**	**Tue 17/08/15**
100	1st fix sprinklers	5 days	Tue 17/06/27	Mon 17/07/03
101	1st fix AC	5 days	Tue 17/06/27	Mon 17/07/03
102	Internal walls (Drywall)	10 days	Tue 17/07/04	Mon 17/07/17
103	Ceilings	5 days	Tue 17/07/18	Mon 17/07/24
104	Doors and glazing	2 days	Tue 17/07/25	Wed 17/07/26
105	Floor finishes	2 days	Thu 17/07/27	Fri 17/07/28
106	Wall finishes	2 days	Thu 17/07/27	Fri 17/07/28
107	Cabinetry	10 days	Mon 17/07/31	Fri 17/08/11
108	2nd fix electrical	2 days	Mon 17/08/14	Tue 17/08/15
109	2nd fix plumbing	2 days	Mon 17/08/14	Tue 17/08/15
110	2nd fix AC	2 days	Mon 17/08/14	Tue 17/08/15



Project: Construction programm
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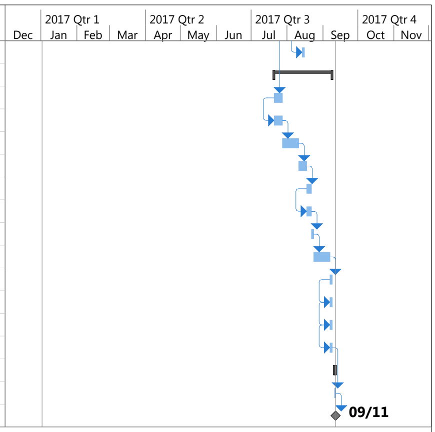
Task		Inactive Summary		External Tasks	
Split		Manual Task		External Milestone	
Milestone		Duration-only		Deadline	
Summary		Manual Summary Rollup		Progress	
Project Summary		Manual Summary		Manual Progress	
Inactive Task		Start-only			
Inactive Milestone		Finish-only			

ID	Task Name	Duration	Start	Finish
111	2nd fix sprinklers	2 days	Mon 17/08/14	Tue 17/08/15
112	**3rd floor**	**36 days**	**Fri 17/07/21**	**Fri 17/09/08**
113	1st fix sprinklers	5 days	Fri 17/07/21	Thu 17/07/27
114	1st fix AC	5 days	Fri 17/07/21	Thu 17/07/27
115	Internal walls (Drywall)	10 days	Fri 17/07/28	Thu 17/08/10
116	Ceilings	5 days	Fri 17/08/11	Thu 17/08/17
117	Doors and glazing	2 days	Fri 17/08/18	Mon 17/08/21
118	Floor finishes	2 days	Fri 17/08/18	Mon 17/08/21
119	Wall finishes	2 days	Tue 17/08/22	Wed 17/08/23
120	Cabinetry	10 days	Thu 17/08/24	Wed 17/09/06
121	2nd fix electrical	2 days	Thu 17/09/07	Fri 17/09/08
122	2nd fix plumbing	2 days	Thu 17/09/07	Fri 17/09/08
123	2nd fix AC	2 days	Thu 17/09/07	Fri 17/09/08
124	2nd fix sprinklers	2 days	Thu 17/09/07	Fri 17/09/08
125	**Close out Phase**	**1 day**	**Mon 17/09/11**	**Mon 17/09/11**
126	Snagging	1 day	Mon 17/09/11	Mon 17/09/11
127	Final handover	0 days	Mon 17/09/11	Mon 17/09/11



Project: Construction programm
Date: Wed 16/10/19

Task		Inactive Summary		External Tasks	
Split		Manual Task		External Milestone	
Milestone		Duration-only		Deadline	
Summary		Manual Summary Rollup		Progress	
Project Summary		Manual Summary		Manual Progress	
Inactive Task		Start-only			
Inactive Milestone		Finish-only			

Appendix C

Additional cost information

C.1 Building foundation and substructure costs

An example of the calculations of the foundation and substructure costs for the short span steel composite structure are shown in Figure C.1 and Figure C.2 below. The calculation procedure is similar for the remainder of the structural alternatives that were considered, with the size of the foundations being adjusted accordingly for each different structure.

FOUNDATION COSTS - STEEL COMPOSITE SHORT SPAN					
PART OF STRUCTURE	**DETAILS**	**UNIT**	**QUANTITY**	**RATE**	**COST**
Excavations and backfilling	**EXCAVATIONS**				
	Excavation in earth not exceeding 2m deep which is applicable				
	to all foundations in the structure				
	* 1.5x1.1x0.3, No. 2	m^3	4.29	R 126.00	R 540.54
	** 2.1x1.6x0.4, No. 8	m^3	37.632	R 126.00	R 4 741.63
	*** 2.9x2.2x0.6, No. 4	m^3	40.832	R 126.00	R 5 144.83
	**** 2.5x1.9x0.5, No. 4	m^3	28.5	R 126.00	R 3 591.00
	WORKING SPACE EXCAVATIONS				
	Back excavations of vertical sides of excavations in earth exceeding 0.5m and not exceeding 1.5m for working space	m^2		R 55.00	
	Back excavations of vertical sides of excavations in earth exceeding 0.5m and not exceeding 1.5m for working space	m^2		R 65.00	
	Area per footing = 2 x depth x (length + breadth)				
	* 1.5x1.1x1.3, No. 2		13.52	R 55.00	R 743.60
	** 2.1x1.6x1.4, No. 8		82.88	R 55.00	R 4 558.40
	*** 2.9x2.2x1.6, No. 4		65.28	R 65.00	R 4 243.20
	**** 2.5x1.9x1.5, No. 4		52.8	R 55.00	R 2 904.00
	RISK OF COLLAPSE EXCAVATIONS				
	Sides of trench and hole excavations < 1.5m deep				
	* 1.5x1.1x0.3, No. 2	m^2	13.52	R 22.26	R 300.96
	** 2.1x1.6x0.4, No. 8	m^2	82.88	R 22.26	R 1 844.91
	**** 2.5x1.9x0.5, No. 4	m^2	52.8	R 26.50	R 1 399.20
	Sides of trench and hole excavations > 1.5m deep				
	*** 2.9x2.2x0.6, No. 4	m^2	65.28	R 26.50	R 1 729.92
	FILLING - Backfilling to trenches, holes				
	* 1.5x1.1x0.3, No. 2	m^3	3.3	R 191.40	R 631.62
	** 2.1x1.6x0.4, No. 8	m^3	26.88	R 191.40	R 5 144.83
	*** 2.9x2.2x0.6, No. 4	m^3	25.52	R 191.40	R 4 884.53
	**** 2.5x1.9x0.5, No. 4	m^3	19	R 191.40	R 3 636.60
	Compaction of ground surface under floors	m^2	488	R 34.80	R 16 982.40
	TOTAL EXCAVATION COST				**R 63 022.17**

Figure C.1: Example of foundation and substructure cost calculations for the short span steel composite structure - Page 1

Concrete	**CONCRETE CAST AGAINST EXCAVATED SURFACES**				
	15MPa/19mm concrete				
	Surface blinding under bases - Use 75mm blinding				
	* 1.5x1.1x0.3, No. 2	m^3	0.2475	R 1 692.30	R 418.84
	** 2.1x1.6x0.4, No. 8	m^3	2.016	R 1 692.30	R 3 411.68
	*** 2.9x2.2x0.6, No. 4	m^3	1.914	R 1 692.30	R 3 239.06
	**** 2.5x1.9x0.5, No. 4	m^3	1.425	R 1 692.30	R 2 411.53
	SURFACE BEDS ON WATERPROOFING				
	Use a 125mm surface bed	m^3	61	R 1 783.29	R 108 780.69
	CONCRETE				
	30MPa, 19mm concrete				
	Bases				
	* 1.5x1.1x0.3, No. 2	m^3	0.99	R 1 549.14	R 1 533.65
	** 2.1x1.6x0.4, No. 8	m^3	10.752	R 1 549.14	R 16 656.35
	*** 2.9x2.2x0.6, No. 4	m^3	15.312	R 1 549.14	R 23 720.43
	**** 2.5x1.9x0.5, No. 4	m^3	9.5	R 1 549.14	R 14 716.83
	Column plinths - 1.5m long				
	* Edge columns - 0.35x0.35, No: 14	m^3	2.5725	R 1 681.95	R 4 326.82
	* Internal columns - 0.4x0.4, No 4	m^3	0.96	R 1 681.95	R 1 614.67
	Total volume of concrete in footings and plinths	m^3	40.0865		
	TOTAL CONCRETE COST				**R 180 830.55**
Reinforcement	**Reinforcement**				
	Reinforcement in bases = 70kg/m^3 (Base and plinth)				
	Various diameter bars	t	2.81	R 10 500.00	R 29 463.58
	Reinforcing mesh for surface bed - Use Ref 193 mesh	t			
	Rate obtained from Gert for Ref 193 Mesh = R34/m^2	m^2	487.5	R 34.00	R 16 575.00
	TOTAL REINFORCEMENT COST				**R 46 038.58**
Formwork - Sides of base only	**Rough formwork to sides**				
	Bases = 2 * depth of base * (length + breadth)				
	* 1.5x1.1x0.3, No. 2	m^2	3.12	R 168.20	R 524.78
	** 2.1x1.6x0.4, No. 8	m^2	23.68	R 168.20	R 3 982.98
	*** 2.9x2.2x0.6, No. 4	m^2	24.48	R 168.20	R 4 117.54
	**** 2.5x1.9x0.5, No. 4	m^2	17.6	R 168.20	R 2 960.32
Formwork - Sides of column plinth only	**Rough formwork to sides of column plinth**				
	Column plinth - 1.5m long, A_{req} = 2*height*(l+b)				
	* Edge columns - 0.35x0.35, No: 14	m^2	29.4	R 168.20	R 4 945.08
	* Internal columns - 0.4x0.4, No 4	m^2	9.6	R 168.20	R 1 614.72
	TOTAL COST OF FORMWORK				**R 18 145.42**
TOTAL FOUNDATION COST					**R 308 036.71**

Figure C.2: Example of foundation and substructure cost calculations for the short span steel composite structure - Page 2

C.2 Building frame costs

Information is provided in this section regarding the cost calculations of the structural frame for each of the structural alternatives that were considered in this study. A breakdown of the calculations that were performed, and all cost items that were considered, is shown in Figure C.3 to C.6 below.

STRUCTURAL FRAME COST - RC FLAT SLAB					
PART OF STRUCTURE	**DETAILS**	**UNIT**	**QUANITY**	**RATE**	**COST**
Floor slab	**30MPa, 19mm concrete**				
	Slabs, including beams and inverted beams	m^3			
	Concrete in 300mm thick floor slab	m^3	146.25	R 1 453.95	R 212 640.19
	Concrete in edge beams - 0.8*0.4*(37.5*2+13*2)	m^3	32.32	R 1 453.95	R 46 991.66
	Volume of concrete in floor and edge beams per floor	m^3	178.57		
	Smooth formwork to soffits and inverted beams				
	Slabs exceeding 250mm and not exceeding 500mm thick and propped < 3.5m high	m^2	488	R 295.77	R 144 335.76
	Inverted beams - (0.8+1.1+0.4)*(37.5*2+13*2)	m^2	232.3	R 547.21	R 127 116.88
	Reinforcement - High tensile steel reinforcement				
	Reinforcing content in floor slab and inverted beams is 90kg/m^3				
	Various diameter bars	t	16.0713	R 10 500.00	R 168 748.65
	Powerfloat surface treatment to finished face of concrete				
	Slabs	m^2	488	R 23.40	R 11 419.20
TOTAL FLOOR COST PER STOREY					R 711 252.34
Columns / Floor	**30MPa, 19mm concrete**				
	Column height = 3.2m as indicated in drawings				
	Internal columns dimensions: 400x400, No: 6	m^3	3.072	R 1 597.68	R 4 908.07
	Internal columns dimensions: 280x400, No: 12	m^3	4.3008	R 1 597.68	R 6 871.30
	Smooth formwork to sides				
	Recatngular columns	m^2	82.944	R 272.55	R 22 606.39
	Reinforcement				
	Reinforcement content in columns is 1% column area				
	Internal columns	t	0.26544	R 10 500.00	R 2 787.12
	Edge columns	t	0.39816	R 10 500.00	R 4 180.68
TOTAL COLUMN COST PER STOREY					R 41 353.56
TOTAL FLOOR AND COLUMN COST PER STOREY					R 752 605.91

Figure C.3: Frame cost calculations for reinforced concrete flat slab building

STRUCTURAL FRAME COST - POST-TENSIONED FLAT SLAB					
PART OF STRUCTURE	DETAILS	UNIT	QUANTITY	RATE	COST
Floor slab	**30MPa, 19mm concrete**				
	Slabs, including beams and inverted beams				
	Concrete in 250mm thick floor slab	m^3	121.875	R 1 453.95	R 177 200.16
	Concrete in edge beams - 0.75*0.4*(37.5*2+13*2)	m^3	30.3	R 1 453.95	R 44 054.69
	Volume of concrete in floor and edge beams	m^3	152.175		
	Smooth formwork to soffits				
	Slabs propped < 3.5m high	m^2	488	R 275.30	R 134 346.40
	Inverted beams - (0.75+1.05+0.4)*(37.5*2+13*2)	m2	222.2	R 547.21	R 121 590.06
	Reinforcement - High tensile steel reinforcement				
	Floors are post-tensioned using high strength steel cables				
	Various diameter reinforcing bars- 10kg/m^2	t	4.88	R 10 500.00	R 51 240.00
	Cables for post-tensioning - 3.5 kg/m^2	t	1.708	R 34 045.00	R 58 148.86
	Dead anchors	No.	100	R 191.51	R 19 151.00
	Live anchors	No.	100	R 170.50	R 17 050.00
	Powerfloat surface treatment to finished face of concrete				
	Slabs	m^2	488	R 23.40	R 11 419.20
TOTAL FLOOR COST PER STOREY					R 634 200.36
Columns	**30MPa, 19mm concrete**				
	Column height = 3.2m as indicated in drawings				
	Internal columns dimensions: 400x400, No: 6	m^3	3.072	R 1 597.68	R 4 908.07
	Internal columns dimensions: 280x400, No: 12	m^3	4.3008	R 1 597.68	R 6 871.30
	Smooth formwork to sides				
	Recatngular columns	m^2	82.944	R 272.55	R 22 606.39
	Reinforcement				
	Reinforcement content in columns is 1% column area				
	Internal columns	t	0.26544	R 10 500.00	R 2 787.12
	Edge columns	t	0.39816	R 10 500.00	R 4 180.68
TOTAL COLUMN COST PER STOREY					R 41 353.56
TOTAL FLOOR AND COLUMN COST PER STOREY					R 675 553.93

Figure C.4: Frame cost calculations for reinforced concrete building with post-tensioned floors

STRUCTURAL FRAME COST - SHORT SPAN COMPOSITE STRUCTURE					
PART OF STRUCTURE	**DETAILS**	**UNIT**	**QUANITY**	**RATE**	**COST**
Floor	**30MPa, 19mm concrete**				
	Slab on Bond-dek sheet ($d_{average}$ * A_{floor}) = (75+65/2) * 488	m^3	52.46	R 1 453.95	R 76 274.22
	Structural steelwork				
	Beams				
	305x102x28 I-section no pre-cambering	t	2.24	R 32 169.05	R 72 058.67
	305x102x28 I-section with pre-cambering	t	4.63	R 33 369.05	R 154 632.18
	406x140x39 I-section with precambering	t	1.46	R 33 369.05	R 48 802.24
	406x140x46 I-section with precambering	t	1.73	R 32 569.05	R 56 181.61
	Total mass of all steel beams in floor	t	10.06		
	Connections - (Assume 10% weight of beams)	t	1.00615	R 37 369.05	R 37 598.87
	Reinforcement - High tensile steel reinforcement				
	Use a 8mm mesh 200x200, Mesh ref 395 (mass/m^2 = 3.95kg/m^2)	t	1.9276	R 10 500.00	R 20 239.80
	Y8-450 trough bars	t	0.43	R 10 500.00	R 4 515.00
	Metal decking and shear studs				
	Use 0.8mm Bond-dek sheet	m^2	488	R 475.00	R 231 800.00
	φ19, 125mm long shear studs welded to flange of beams.	No.	1460	R 50.00	R 73 000.00
	Powerfloat surface treatment to finished face of concrete				
	Slabs	m^2	488	R 23.40	R 11 419.20
	Fire protection to beams - See fire cost calculations for details				
	Fire protection to floor system for a 1 hour fire rating with SPM				R 44 079.42
	Use vermiculite spray applied to beams concealed in floor				
TOTAL FLOOR COST PER STOREY					**R 830 601.20**
Columns / floor	**Structural steelwork**				
	Floor-to-floor height of 3.64m as indicated in drawings				
	152x152x30 H-section	t	0.2184	R 31 569.05	R 6 894.68
	203x203x46 H-section	t	2.0094	R 31 569.05	R 63 434.85
	203x203x71 H-section	t	1.0338	R 30 169.05	R 31 188.76
	Total mass of steel for columns / storey	t	3.2616		
	Connections - (Assume 5% weight of columns)	t	0.16308	R 37 369.05	R 6 094.14
	Fire protection to columns - See fire cost calculations for details				
	Vermiculite spray for a 1 hour fire rating - All columns protected				R 22 496.95
	Cost of light gypsum board to frame column	m2	42.768	R 60.00	R 2 566.08
TOTAL COLUMN COST PER STOREY					**R 132 675.47**
TOTAL FLOOR AND COLUMN COST PER STOREY					**R 963 276.67**

Figure C.5: Frame cost calculations for short span steel framed building with composite Bond-Dek floors

FRAME COST - SHORT SPAN HOLLOWCORE STRUCTURE					
PART OF STRUCTURE	**DETAILS**	**UNIT**	**QUANITY**	**RATE**	**COST**
Floor	**30MPa, 19mm concrete**				
	40mm levelling screed on hollowcore units	m^3	19.52	R 1 453.95	R 28 381.10
	Verify this rate - Not sure of exact rate to use for floor screed				
	Structural steelwork				
	Beams				
	203x133x25 I-section, no precambering, Girder A,B,C	t	2.81	R 32 169.05	R 90 475.45
	305x165x40 I-section, no precmbering, Edge beam A-B	t	0.40	R 31 369.05	R 12 547.62
	356x171x45 I-section, no precambering, Composite beam A-B	t	0.90	R 31 369.05	R 28 232.15
	406x178x60 I-section with precambering, Edge beam B-C	t	0.96	R 32 569.05	R 31 266.29
	406x178x60 I-section with precambering, Composite beam B-C	t	1.92	R 32 569.05	R 62 532.58
	Total mass of all steel beams in floor	t	6.99		
	Connections - (Assume 10% weight of beams)	t	0.69925	R 37 369.05	R 26 130.31
	Hollowcore floors slabs - See TopFloor quote				
	Total costs per floor level (including crane)	m^2	488	R 621.26	R 303 174.88
	Reinforcement - High tensile steel reinforcement				
	Y16 transverse reinforcement for composite beams and slabs	t	0.6636	R 10 500.00	R 6 967.80
	Use mesh ref 193 (1.93 kg/m^2) - Light mesh to control cracking	t	0.940875	R 10 500.00	R 9 879.19
	Shear studs welded to flange of top beam				
	φ19, 125mm long shear studs welded to flange of beams.	No.	390	R 50.00	R 19 500.00
	Powerfloat surface treatment to finished face of concrete				
	Slabs	m^2	488	R 23.40	R 11 419.20
	Fire protection to beams - See fire cost calculations for details				
	Fire protection to floor system for a 1 hour fire rating				R 55 308.95
	Use vermiculite spray applied to beams concealed in floor				
TOTAL FLOOR COST PER STOREY					**R 685 815.51**
Columns / floor	**Structural steelwork**				
	Columns - Use a floor-to-floor height of 3.8m as indicated in drawings				
	152x152x37 H-section	t	0.2812	R 31 569.05	R 8 877.22
	203x203x46 H-section	t	1.0488	R 31 569.05	R 33 109.62
	203x203x52 H-section	t	0.3952	R 31 569.05	R 12 476.09
	203x203x71 H-section	t	1.0792	R 30 169.05	R 32 558.44
	203x203x89 H-section	t	1.3528	R 30 169.05	R 40 812.69
	Total mass of steel for columns / storey	t	4.1572		
	Connections - (Assume 10% weight of columns)	t	0.41572	R 37 369.05	R 15 535.06
	Fire protection to columns - See fire cost calculations for details				
	Vermiculite spray for a 1 hour fire rating - All columns protected				R 23 789.00
	Cost of light gypsum board to frame column	m^2	42.768	R 60.00	R 2 566.08
TOTAL COLUMN COST PER STOREY					**R 169 724.20**
TOTAL FLOOR AND COLUMN COST PER STOREY					**R 855 539.71**

Figure C.6: Frame cost calculations for short span steel framed building with hollowcore floor units

C.3 Fire protection costs

Information is provided in this section relating to the calculation of the fire protection costs for the steel structural alternatives considered in this study.

C.3.1 Specifying required thickness of fire protection material

The methodology that was followed when calculating the required thickness of passive fire protection was discussed in Section 3.6.5. Figure C.7 provides an example of the product information that was used when specifying the required intumescent paint thickness.

Required Intumescent Dry Film Thickness (dft) in millimetres (mm) of Interchar 1190 for a fire resistance period of 60 minutes for I- and H- section beams.

Product Name	Section factor (Hp/A)	Design temperature (°C)									LPCB Ref. No
		350	400	450	500	550	600	620	650	700	
Interchar 1190		Intumescent Dry Film Thickness (dft) mm									1111a/01
	30	0.851	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	35	0.890	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	40	0.929	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	45	0.967	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	50	1.006	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	55	1.045	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	60	1.084	0.770	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	65	1.123	0.791	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	70	1.161	0.815	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	75	1.200	0.839	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	80	1.239	0.863	0.770	0.770	0.770	0.770	0.770	0.770	0.770	
	85	1.278	0.888	0.781	0.770	0.770	0.770	0.770	0.770	0.770	
	90	1.317	0.912	0.798	0.770	0.770	0.770	0.770	0.770	0.770	
	95	1.355	0.936	0.816	0.770	0.770	0.770	0.770	0.770	0.770	
	100	1.394	0.960	0.833	0.770	0.770	0.770	0.770	0.770	0.770	

Figure C.7: Product information for calculating the thickness of intumescent paint required

C.3.2 Cost of intumsecent paint and vermiculite spray

Figure C.8 shows the relationship between the required dry film thickness of intumescent paint versus the cost. The intumescent paint costs increases linearly as the dry film thickness increases. The cost were therefore approximated by a linear trend line and used to calculate the cost of applying different thickness intumescent paint.

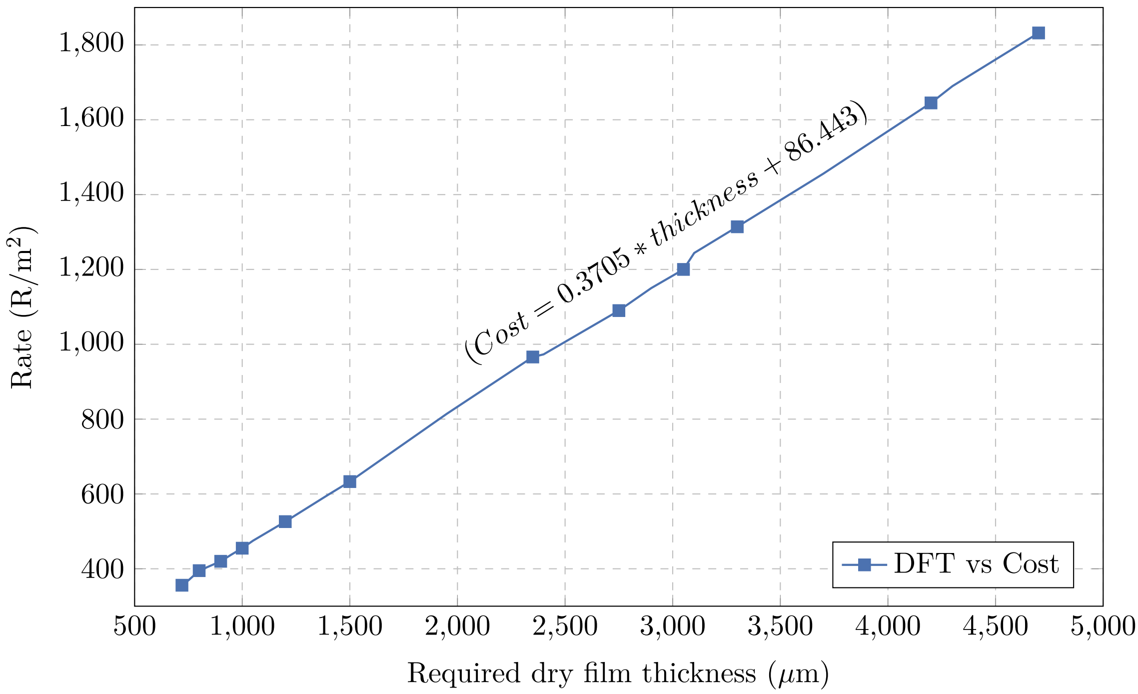


Figure C.8: Relationship between the dry film thickness of intumescent paint versus the cost per square metre of steel painted

Figure C.9 shows the relationship between the required thickness of vermiculite spray versus the cost. As for intumescent paint the cost of vermiculite spray increases linearly as the thickness increases.

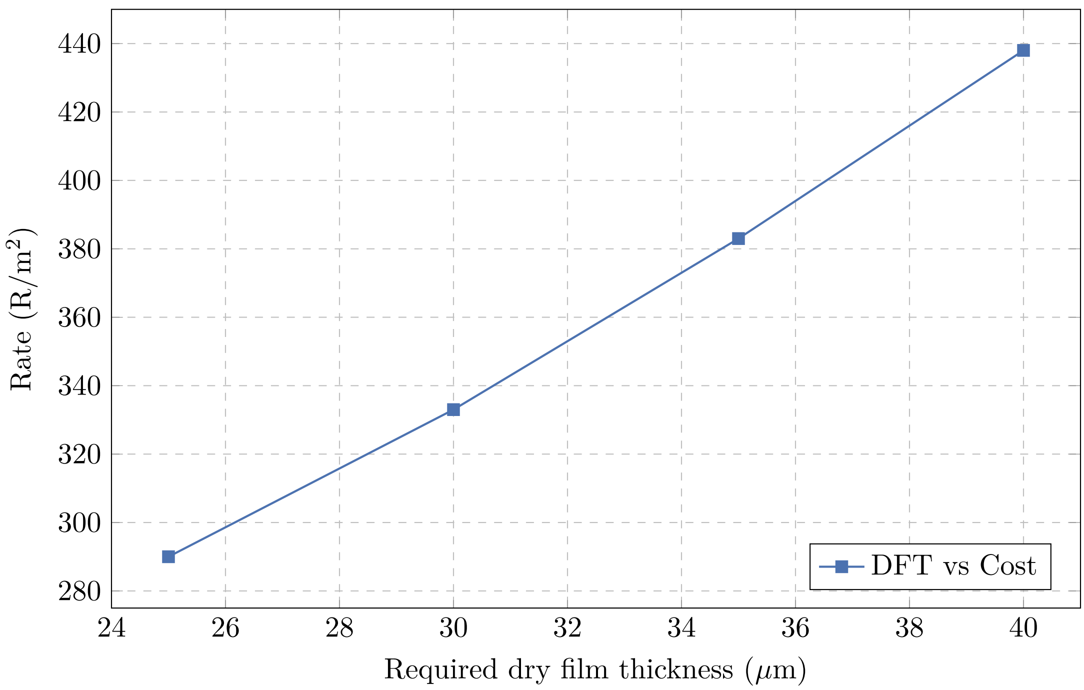


Figure C.9: Relationship between the thickness of vermiculite spray versus the cost per square metre of steel painted

C.3.3 Cost calculations for passive fire protection

Figure C.10 provides an example of how the fire protection costs were calculated for the short span steel composite structure for a 60 minute fire rating, using vermiculite paint. The calculation procedure is similar for the other steel structures, fire ratings and materials with the steel sizes, material thickness and cost rate being adjusted accordingly.

		BOND-DEK SLAB SHORT SPAN STRUCUTRE																
		DETAILS						MATERIAL				DATE		CLIENT :-		Michael Drennan		
		Short span composite building						Vermiculite 60 mins				2016/07/21		TENDER No. :-		T 4324		
		REF.	ITEM	WEB	FLG.	KG	No. SIDES	$\frac{H_p}{A}$	THK. DFT	LENGTH STEEL	AREA / m^2	RATE R / m^2	VALUE R/LENGTH	TOTAL QTY.	TOTAL VALUE	ACTUAL QTY.	ACTUAL VALUE	TOTAL m^2
ALL BEAMS PROTECTED	Floor beams / storey	Girder A	Beam	305	102	28	3	257	25	7.50	0.92	290.00	1 992.30	5	R 9 961.50			34
		Girder B	Beam	406	140	46	3	210	25	7.50	1.23	290.00	2 679.60	5	R 13 398.00			46
		Girder C	Beam	406	140	39	3	248	25	7.50	1.23	290.00	2 679.60	5	R 13 398.00			46
		5m-EDGE	Beam	305	102	28	3	257	25	5.00	0.92	290.00	1 328.20	2	R 2 656.40			9
		5m-INT	Beam	305	102	28	3	257	25	5.00	0.92	290.00	1 328.20	14	R 18 594.80			64
		8m-EDGE	Beam	305	102	28	3	257	25	8.00	0.92	290.00	2 125.12	2	R 4 250.24			15
		8m-INT	Beam	305	102	28	3	257	25	8.00	0.92	290.00	2 125.12	14	R 29 751.68			103
														47	R 92 010.62			317
SLAB PANEL METHOD	Floor beams / storey	Girder A	Beam	305	102	28	3	257	25	7.50	0.92	290.00	1 992.30	5	R 9 961.50			34
		Girder C	Beam	406	140	39	3	248	25	7.50	1.23	290.00	2 679.60	5	R 13 398.00			46
		5m-EDGE	Beam	305	102	28	3	257	25	5.00	0.92	290.00	1 328.20	2	R 2 656.40			9
		5m-INT	Beam	305	102	28	3	257	25	5.00	0.92	290.00	1 328.20	4	R 5 312.80			18
		8m-EDGE	Beam	305	102	28	3	257	25	8.00	0.92	290.00	2 125.12	2	R 4 250.24			15
		8m-INT	Beam	305	102	28	3	257	25	8.00	0.92	290.00	2 125.12	4	R 8 500.48			29
														22	R 44 079.42			152
		Ground - 1																
COLUMNS BELOW SPLICE POSITION	Columns G-1	A 1 / 6	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		A 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
		B 1/6	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	2	R 2 571.44			9
		B 2/3/4/5/	Col.	203	203	71	4	135	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
		C 1/6	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	2	R 2 571.44			9
		C 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
														18	R 22 496.95			
		Level 1- 2																
	Columns 1-2	A 1 / 6	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		A 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
		B 1/6	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	2	R 2 571.44			9
		B 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
		C 1/6	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	2	R 2 571.44			9
		C 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
														18	R 22 496.95			
		Level 2 - 3																
COLUMNS BELOW SPLICE POSITION	Columns 2-3	A 1 / 6	Col.	152	152	23	4	311	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		A 2/3/4/5/	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	4	R 3 850.83			13
		B 1/6	Col.	152	152	37	4	193	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		B 2/3/4/5/	Col.	203	203	46	4	208	25	3.64	1.22	290.00	1 285.72	4	R 5 142.88			18
		C 1/6	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		C 2/3/4/5/	Col.	152	152	37	4	193	25	3.64	0.91	290.00	962.71	4	R 3 850.83			13
														18	R 18 620.78			
		Level 3 - 4																
	Columns 3-4	A 1 / 6	Col.	152	152	23	4	311	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		A 2/3/4/5/	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	4	R 3 850.83			13
		B 1/6	Col.	152	152	37	4	193	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		C 1/6	Col.	152	152	30	4	239	25	3.64	0.91	290.00	962.71	2	R 1 925.41			7
		C 2/3/4/5/	Col.	152	152	37	4	193	25	3.64	0.91	290.00	962.71	4	R 3 850.83			13
														14	R 13 477.90		Total area	1 204

Figure C.10: Calculation procedure for fire protection costs using vermiculite spray to achieve a 1 hour fire rating

C.4 Cash flow development calculations

This section provides an example of the calculation procedure to determine the cash flow development for each of the structural alternatives. An examples of the calculation procedure that was followed is shown in Table C.1.

Table C.1: Example of cash flow development for a steel and concrete structural alternative

Structure	Short span hollowcore building	
Construction time	7	months

	Month	Date	Property costs	Funding costs	Construction cost	Professional fees	Local authority costs	Promotional costs	Sundries	Total capital investment	Cumulative cost	Interest on borrowed money	Cumulative interest	Income	Cumulative interest + capital
			R	R	R	R	R	R	R	R	R	R	R	R	R
			2 722 600.00	-	18 949 960.00	2 987 243.80	150 000.00	100 000.00	124 549.02	25 034 352.82		940 778.07		-284 600.00	
tender	0	Feb-17	2 722 600.00	-	-	1 792 346.28	50 000.00	-	-	4 564 946.28	4 564 946.28	38 140.84	38 140.84	-	4 603 087.12
start	1	Mar-17	-	-	833 798.24	170 699.65	14 285.71	14 285.71	17 792.72	1 050 862.03	5 615 808.31	46 920.95	85 061.79	-	5 700 870.10
	2	Apr-17	-	-	2 141 345.48	170 699.65	14 285.71	14 285.71	17 792.72	2 358 409.27	7 974 217.58	66 625.83	151 687.62	-	8 125 905.20
	3	May-17	-	-	3 240 443.16	170 699.65	14 285.71	14 285.71	17 792.72	3 457 506.95	11 431 724.53	95 513.84	247 201.46	-	11 678 925.99
	4	Jun-17	-	-	4 159 516.22	170 699.65	14 285.71	14 285.71	17 792.72	4 376 580.01	15 808 304.55	132 080.85	379 282.30	-	16 187 586.85
	5	Jul-17	-	-	3 363 617.90	170 699.65	14 285.71	14 285.71	17 792.72	3 580 681.69	19 388 986.24	161 998.00	541 280.30	-	19 930 266.54
	6	Aug-17	-	-	3 174 118.30	170 699.65	14 285.71	14 285.71	17 792.72	3 391 182.09	22 780 168.33	190 331.85	731 612.15	-	23 511 780.48
	7	Sep-17	-	-	2 037 120.70	170 699.65	14 285.71	14 285.71	17 792.72	2 254 184.49	25 034 352.82	209 165.92	940 778.07	-	25 975 130.89
end	8	Oct-17	-	-	-	-	-	-	-	-	25 034 352.82	209 165.92	1 149 943.98	-284 600.00	25 899 696.80

Structure	RC flat slab building	
Construction time	8	months

	Month	Date	Property costs	Funding costs	Construction cost	Professional fees	Local authority costs	Promotional costs	Sundries	Total capital investment	Cumulative cost	Interest on borrowed money	Cumulative interest	Income	Cumulative interest + capital
			R	R	R	R	R	R	R	R	R	R	R	R	R
			2 722 600.00	-	19 208 110.00	3 027 257.05	150 000.00	100 000.00	126 039.84	25 334 006.89		1 073 693.73		-	
tender	0	Feb-17	2 722 600.00	-	-	1 816 354.23	50 000.00	-	-	4 588 954.23	4 588 954.23	38 341.43	38 341.43	-	4 627 295.66
start	1	Mar-17	-	-	739 512.24	151 362.85	12 500.00	12 500.00	15 754.98	931 630.07	5 520 584.30	46 125.34	84 466.77	-	5 605 051.07
	2	Apr-17	-	-	1 757 542.07	151 362.85	12 500.00	12 500.00	15 754.98	1 949 659.90	7 470 244.19	62 415.05	146 881.82	-	7 617 126.02
	3	May-17	-	-	2 468 242.14	151 362.85	12 500.00	12 500.00	15 754.98	2 660 359.97	10 130 604.16	84 642.77	231 524.60	-	10 362 128.76
	4	Jun-17	-	-	3 639 936.85	151 362.85	12 500.00	12 500.00	15 754.98	3 832 054.68	13 962 658.84	116 660.19	348 184.78	-	14 310 843.62
	5	Jul-17	-	-	3 284 586.81	151 362.85	12 500.00	12 500.00	15 754.98	3 476 704.64	17 439 363.48	145 708.60	493 893.38	-	17 933 256.86
	6	Aug-17	-	-	2 958 048.94	151 362.85	12 500.00	12 500.00	15 754.98	3 150 166.77	20 589 530.25	172 028.73	665 922.11	-	21 255 452.36
	7	Sep-17	-	-	2 689 135.40	151 362.85	12 500.00	12 500.00	15 754.98	2 881 253.23	23 470 783.48	196 102.05	862 024.16	-	24 332 807.65
	8	Oct-17	-	-	1 671 105.57	151 362.85	12 500.00	12 500.00	15 754.98	1 863 223.40	25 334 006.89	211 669.57	1 073 693.73	-	26 407 700.62
end	9	Nov-17	-	-	-	-	-	-	-	-					

C.5 Breakdown of structural steel cost

Information is provided in this section regarding how the rates were derived for various steel sections used in the study. This information was obtained from a discussion with the CEO of a steel fabrication company regarding how steel costs are developed during a project. The minutes from this meeting can be seen in Appendix D. During this discussion steel costs obtained from recently tendered steel projects were broken down to reveal the components that make up the total cost of steel. These components are shown in Table C.2. From Table C.2 it is clear that there are several components that make up the total steelwork cost during a project. These components include aspects such as material costs, fabrication, erection and any coatings that are required. It is therefore important that all these aspects are considered when deriving costs for steelwork during a project.

Table C.2: Breakdown of cost of steelwork

Rate for galvanized steelwork from De Leeuw	R 37 619.05 / ton
Galvanizing	R 6 000.00 / ton
Paint (Around R100/m^2, approx 35 m^2/ton)	R 3 500.00 / ton
Total for galvanizing and paint	R 9 500.00 / ton
Detailing	R 800.00 / ton
Material and waste (10 %)	R 13 000.00 / ton
Allowance for bolts (1.5 % of steel mass)	R 195.00 / ton
Fabrication	R 5 000.00 / ton
Delivery to site	R 200.00 / ton
Cost of erecting steelwork	R 5 000.00 / ton
Total cost price for steel fabricator	R 24 195.00 / ton
Profit	R 3 924.05 / ton

In addition to discussing the total cost rate for steelwork, the fabrication costs for different steel elements were discussed. The results of this discussion are shown in Table C.3 below.

Table C.3: Breakdown of fabrication costs for various steel elements

Section details	Mass [kg/m]	Hour / ton range	Fabrication R / ton
Columns	40–60	14 – 17	R 6 200.00
	60-80	10 –14	R 4 800.00
Beams	25–40	16 – 18	R 6 800.00
	40–60	14 – 16	R 6 000.00
Angle bracing	Varies	8 – 10	R 3 600.00
Circular Hollow Sections	Varies	18 – 22	R 8 000.00
Connections	Varies	30	R 12 000.00
Precambering	Varies	2 – 4	R 1 200.00

Table C.3 shows the fabrication costs for different steel sections and masses. Fabrication costs differ depending on the profile of the steel section. Bracing elements for example, require little fabrication effort and as such have a low fabrication cost compared to other steel sections. Additionally, fabrication costs differ depending on the mass of the section with the fabrication cost being reduced as the mass of the steel increases. It is therefore important to develop steel rates independently for each of the steel sections in a project to ensure that the costs are realistic. Furthermore, Table C.3 reveals how the lowest mass of steel may not always provide the cheapest solution. Once aspects such as fabrication and fire protection costs are considered it could potentially be more economical to make use of a slightly heavier section.

Appendix D

Meeting minutes

During the course of this thesis numerous meetings and discussions were held with industry professionals. The purpose of these meetings was to obtain information and insight into various topics that could assist with the research conducted during the course of this study. This appendix presents a summarised minutes of these meetings that took place, and reveals some of the information that was gained. An ethical clearance application was submitted and permission was obtained from the Research Ethics Committee at Stellenbosch University to interview the participants in this study.

D.1 Union Steel

Union Steel is one of the leading structural and mechanical contractors of structural steelwork in Cape Town. They have many years of experience in procuring, fabricating and delivering steelwork for a wide range of projects. Information is obtained regarding the steel framed structures considered in the study is obtained.

Meeting 1

Date	10 March 2016
Occupation of participant	CEO
Purpose of meeting	Discuss various aspects regarding the design of the steel framed structures

Topics discussed and information gained:

- *Floor layout* - The floor layout that has been developed was considered to be representative of a typical structure and can be achieved with steel construction.
- *Bracing members* - The use of angle sections to provide lateral stability in the form of braced bays is considered to be an effective method of providing lateral stability.
- *Columns* - The splice positions that have been identified for the steel framed structures

are considered to be acceptable. Columns frequently span over two stories and it is even possible to make them span over three stories if this is required.

- *Pre-cambering* - If deflection is governing the design then pre-cambering is a viable option.

Meeting 2

Date	1 June 2016
Occupation of participant	CEO
Purpose of meeting	Discuss various aspects regarding the design of the steel framed structures

Topics discussed and information gained:

- *Secondary beams* - Initially beams were used at mid-span in the long span hollowcore structure due to the very high beam loads that are experienced. However, this is not particularly economic due to the fact that a large number of beams were required and the girders were required to carry load. It makes more sense to use a plate girder to resist the moment and do away with any secondary beams.
- *Fire protection* - Discussed methods to provide fire protection and important considerations when considering the fire protection of steel members.
- *Floor vibrations* - Floor vibrations are a critical serviceability consideration and if not given sufficient consideration have been shown to be problematic.
- *Erection rates* - A typical erection rate of 3-4 steel members per hour was recommended as being realistic. This equates to approximately 20-25 members being able to be erected per day.

Meeting 3

Date	4 July 2016
Occupation of participant	CEO
Purpose of meeting	Discuss cost and programme aspects regarding steel framed structures

Topics discussed and information gained:

- *Cost breakdown of structural steel rates* - Discussed how steel costs are derived considering a rate obtained from a recently tendered steel project.

- *Fabrication costs* - Discussed fabrication costs for various steel elements and masses. The conclusion is that lighter members have lower fabrication costs per ton of steelwork.
- *Lowest weight not always cheapest* - The lowest weight of steelwork does not always provide the cheapest solution. For example, a lighter beam will save on material costs but once the increased cost of fire protection and fabrication are considered, it may not prove to be the most economical solution. Furthermore using fewer, heavier members reduces construction time which will also have costs benefits associated with it.
- *Erection rates and costs* - An erection cost of R 5000 / day is recommended.
- *Erection rate* - From the erection rates that were assumed it was decided that it would take approximately 3 weeks to erect the entire steel frame for the composite building. The time can be reduced further for the hollowcore structure due to the absence of the secondary beams.
- *Cost rates* - Rates for cost estimation purposes were obtained for Bond-Dek sheeting as well as for the welding of shear studs.
- *General* - General considerations involved with the design and implementation of structural steel for more multi-storey office buildings were discussed. Some of the opinions of the participant regarding the South African building industry were the following:
  - The cost advantages that can be associated with a shorter construction period are very rarely considered when performing comparisons between steel and concrete structures.
  - The South African construction industry is reluctant to change. There is certainly potential for the increased use of alternative building methods to reinforced concrete, such as structural steel with a light steel frame, but in order for it to be implemented successfully it requires a shift in mindset.
  - There is currently a knowledge gap among professionals in the South African construction industry with regards to multi-storey building design using structural steel. This knowledge gap, and lack of familiarity means that steel is not always given any consideration when selecting a frame material during a project.

D.2 BICS Fireproofing

BICS Fireproofing are a company specializing in the supply of passive fire protection for steel frames structures. BICS are a South African company with 24 years of experience in the fire protection and fireproofing industry.

Date	10 March 2016
Occupation of participant	Managing director

Purpose of meeting	Discuss a range of aspects pertaining to the passive fire protection of steel framed structures

- *Passive fire protection systems* - Discussed systems that are currently used to provide passive fire protection. The primary systems that are used are intumescent paints, vermiculite sprays and boards.
  - Vermiculite spray - Vermiculite sprays are able to achieve high fire ratings (>2 hours) if required. They are cheap but messy to apply ,and not aesthetically pleasing. If used on floor beams then suspended ceiling will be required so that beams are not visible.
  - Intumescent paint - Intumescent paints are a good option, particularly for relatively short fire resistance periods (60 minutes or less). As the fire rating increases, the cost of the paint increases significantly.
  - Boards - Viable option and can provide an effective solution, but at present not manufactured in South Africa.
- *Construction programme* - On-site application follows erection sequence of steel structure, and is able to match steel erection rate so as not to cause time delays.
- *On-site vs Off-site application* - A challenge of off-site application is during erection the paint is scratched / rubbed off and compromises the desired aesthetic finish of the beam.
- *Early collaboration and involvement* - Importance of early collaboration between designers and fire engineers was stressed. Very important to be considered in conjunction with primary design procedures, and not as a secondary trade following the completion of the design.

D.3 UWP Consulting

UWP Consulting are an experienced South African consulting engineering company, with many years of experience in the design of multi-storey commercial structures.

Date	13 April 2016
Occupation of participant	Director
Purpose of meeting	Discuss various aspects regarding multi-storey commercial structures in SA, and in particular the concrete framed structural alternatives that are being considered in the study.

Topics discussed and information gained:

- *Floor layout* - Discussed building layout and realistic for a multi-storey office building in South Africa.
- *Service zone* - 500 mm is sufficient for placement of services and ceiling. 500 mm gives 150 mm space for ceiling and 350 mm for services in building which should be sufficient.
- *Foundation sizes and reinforcement* - Agreed upon using a bearing capacity of 200 kPa for the soil for all structures. Discussed information regarding foundation design and reinforcement contents. A reinforcement content of 70 kg/m^3 was recommended for cost estimation purposes.
- *Preliminary column sizes* - For edge columns use 280x400 columns were calculated to be sufficient with 400x400 columns being required for internal column sections. Reinforcing in the column can be calculated based on 1% of the column area.
- *Reinforced concrete flat slab structure* - The following information was recommended regarding the reinforced concrete flat slab structure:
  - Slab thickness = 300mm
  - Reinforcement content in slab = 27 kg/m^2
  - Concrete strength = 30 MPa
- *Post-tensioned flat slab structure* - The following information was recommended regarding the post-tensioned concrete flat slab structure:
  - Slab thickness = 250 mm
  - Reinforcement content in slab = 10 kg/m^2
  - Cable content in slab = 3.5 kg/m^2
  - Unbonded construction is recommended
  - Cable diameter = 15.2 mm
  - Concrete strength = 30 MPa

D.4 TopFloor

TopFloor are a company specialising in precast hollowcore concrete floor panels.

Date	11 May 2016
Occupation of participant	Sales associate

Purpose of meeting	Discuss various aspects regarding the use of precast hollow-core slabs in steel framed structures.

Topics discussed and information gained:

- *Lead-in time* - Lead in time for hollowcore slabs is typically 2 weeks.
- *Erection rates* - An erection rate of $500/\mathrm{m}^2$ is achievable on a site with good access.
- *Bearing on steel beam* - A minimum bearing width of 50 mm was advised for the hollowcore units resting on the steel beams. For composite construction it was advised that the flange width of the beam should be at least 171 mm.
- *Reluctance to change* - Many design engineers are unfamiliar or reluctant to specify precast hollowcore slabs.
- *Early collaboration* - The importance of early collaboration between the client, engineer and contractor was stressed. Early collaboration allows for the full benefits of hollowcore slabs to be realised, through choosing spans and building layouts that will allow the most efficient solution to be realised.
- *Hollowcore suitable for low to medium rise structures* - TopFloor believe that hollowcore slabs are ideally suited to low to medium rise multi-storey structures and it is an area where they predict growth in the future.

D.5 De Leeuw Quantity Surveyors

De Leeuw Quantity Surveyors are a quantity surveying company with experience in many commercial construction projects in South Africa.

Date	5 May, 1 July and 13 September 2016
Occupation of participant	Quantity Surveyor specialising in cost estimating
Purpose of meeting	Discuss aspects regarding the cost comparison in the study and to obtain costs from current projects.

Meeting 1 , 2 & 3

Topics discussed and information gained:

- *Construction costs* - Obtained cost information regarding several recently tendered projects. The cost information reflected projects of a similar nature to the structure considered in this study, and cost information was obtained for both steel and concrete structures.

- *Non-structural component cost* - Obtained costs for a wide range of non-structural components including finishes, mechanical and electrical services, roof construction, plumbing and wetpoints and the cost of a lift.
- *Income* - An income of R 150/m^2 of office space was recommended as being realistic.
- *Additional cost components* - Additional costs for land, fencing and security as well as parking around the perimeter of the structure were obtained. Obtained cost rates for reinforcing mesh in steel structural alternatives.
- *Reviewed costs* - Discussed cost estimations that have been developed thus far and refined cost estimates where necessary.
- *P&G costs* - Discussed P&G costs for steel and concrete buildings and recommended typical percentages for each of these structures.
- *Interest rate* - An interest rate of 10.5 % was recommended as the current rate in South Africa for development loans.

D.6 DVPM

DVPM are a project management company specialising in construction projects. DVPM provides project and construction management services for a wide range of construction projects including commercial, industrial, housing and specialist projects, to name a few.

Date	8 July, 22 July 2016
Occupation of participant	Managing director
Purpose of meeting	Discuss aspects regarding the construction programme for the steel and concrete structural alternatives

Meeting 1 & 2

Topics discussed and information gained:

- *Holistic approach* - It was recommended that a holistic approach needs to be taken when developing cost comparisons. It is important that an approach is developed that considers all aspects influencing the overall cost-effectiveness of a structure and not simply the cost of the structure.
- *Method statement* - The development of a construction method statement was discussed and that the development of a method statement would allow the construction programme to be developed more easily.
- *Structural engineers provide resistance* - In the participant's experience, the people who

are the most reluctant to make a transition away from reinforced concrete to structural steel are structural engineers. This could be attributed to the fact that engineers are familiar with concrete and are not willing to shift their mind-set to a construction method that is foreign to them.

- *Logistics* - Logistics and buildabilty are an important consideration in a project. A construction site with limited access may be more suited to a less site intensive construction method, such as steel construction.

- *Lack of knowledge* - In the participant's experience, there is currently very few South African design engineers with an extensive knowledge of steel construction and design. This is an obstacle inhibiting the growth of steel construction in South Africa.

D.7 Isipani Construction

Isipani Constrction are a South African construction company with extensive experience in the construction of a variety of structures, including multi-storey commercial structures.

Date	17 May 2016
Occupation of participant	Director
Purpose of meeting	Discuss practical aspects related to the construction of steel and concrete multi-storey commercial structures

Topics discussed and information gained:

- *Construction rates* - It was recommended that the steel framed structure would be able to reduce the duration of the construction programme by approximately one month compared to the concrete framed structures.

- *Time-related costs* - The early completion of the steel structure would lead to time-related savings with regards to being able to earn income at an earlier stage and a reduction in P&G costs. P&G costs were discussed and it was mentioned that the majority of these costs depend on the duration of the construction programme, so a shorter construction period would lead to cost savings in this area.

- *Income* - An income of R 150/m^2 was recommended as being realistic for a typical office structure.

- *Cost estimation* - Performed a preliminary cost analysis of a steel and concrete structure and identified important components to consider when developing the comparison.